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Railway Engineering-Maintenance Suppliers Association, Inc.
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(703) 241-8514
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D. E. Staplin, Editor

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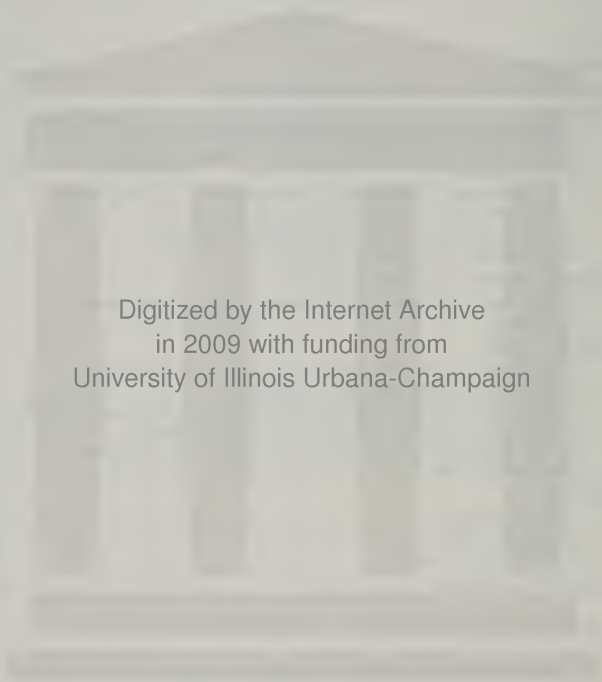

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LAFAYETTE RAILROAD RELOCATION PROJECT

By: C. E. Gullakson*

(This is a synopsis of the presentation made to the AREA Chicago Technical Conference on March 22, 1995.)

Introduction

This project is a cooperative effort between the city of Lafayette, the Indiana Department of Transportation, CSX Transportation and Norfolk Southern to relocate its lines into a grade crossing free corridor for the purpose of eliminating highway/railroad grade crossings within the city.

Geographical Background

To give you some historical and geographical background concerning the area and the project, the city of Lafayette is located in west central Indiana having a population of approximately 45,000. The area is the home of Purdue University and several large manufacturing plants which include Caterpillar, A. E. Staley, Saburu-Isuzu, Eli Lilley, and Wabash-National, to name a few.

Currently, four (4) freight railroads have lines which serve the area and intersect within the city limits. These include CSX Transportation, Norfolk Southern, Conrail, and the Kankakee, Beaverville and Southern. In addition, Amtrak's "Cardinal" and "Hoosier State" passenger trains make a stop in the city.

The segment of CSX Transportation's line through Lafayette was constructed by the New Albany & Salem Railroad in 1853. To promote the line's construction through Lafayette, city officials granted the railroad the right to construct its mainline down the center of fourteen blocks of Fifth Street, one of the city's downtown streets.

Through various mergers and acquisitions, this railroad eventually became part of the Monon Railroad's mainline route between Chicago and Louisville. Later, the line became part of CSX Transportation's railroad system.

Also, during the mid-1800's, predecessors to today's Norfolk Southern and Conrail railroad companies constructed lines into Lafayette's downtown area crossing many streets and roads at grade.

Of course, at the time of construction and for many decades thereafter, conflicts between train traffic and the horse and buggy and pedestrian traffic common in that era, were not an issue of concern.

Project History

However, as the use of automobiles increased in the early 1900's, congestion and accidents prompted several studies by the Monon Railroad and the city to determine alternatives for relocating the Monon's main track out of Fifth Street and the city's downtown (see cover photo).

Beginning in 1969, plans for grade separation structures, new rail routes by-passing the city, and new rail corridors through various parts of the city were studied.

After several years of studies and negotiations between the various railroads and the city, a conceptual plan for relocating CSX and Norfolk Southern's mainlines into a common corridor along the Wabash River was agreed upon.

For CSX, the new corridor begins at its Lafayette Yard at the north end of the city and swings southwestward between the downtown area and the river along the route of the former Conrail/ Norfolk Southern right-of-way before tying back into its existing right-of-way near Lafayette Junction.

*Senior Project Engineer, CSX Transportation

The relocation of Norfolk Southern's Ft. Wayne-St. Louis mainline begins at its yard on the northeast side of the city. It will then head west and follow an alignment alongside CSX near its Lafayette Yard and share a common corridor with CSX until it ties back into its existing alignment near Lafayette Junction.

Norfolk Southern's former Muncie Branch, which now also serves as a Conrail route and a connection to the Kankakee, Beaverville and Southern, also required relocation.

With the execution of general agreements between the city, CSX and Norfolk Southern, the city proceeded with detailed design of the proposed corridor and with applications for state and federal funding. Raymond Kaiser Engineers developed the initial design, and Howard Needles Tammen & Bergendoff developed the final construction plans.

To accommodate the availability of funding, the overall relocation project was divided into five stand-alone phases.

Inasmuch as state and federal funding is being used for the project's construction, the Indiana Department of Transportation has administered the numerous construction contracts involved.

Phase 1—Wabash Avenue Underpass

The first phase of the project was the "Wabash Avenue Underpass" phase which involved the relocation of 1.3 miles of Norfolk Southern's Muncie Branch, roadbed grading for the future CSX and Norfolk Southern tracks, construction of the bridge abutments and retaining walls for 4 railroad bridges, and construction of the superstructure for 1 of the 4 abutments.

When this phase was completed, 4 street crossings were closed. Construction was begun in 1986 and completed in 1987.

The Muncie Branch track was relocated onto the new structure over Wabash Avenue, with an alignment and abutments to accommodate the relocation of the CSX mainline under Phase 4 and Norfolk Southern's double-track mainline under Phase 5 (see Figure 1).

Figure 1. NS Wabash Avenue Bridge.

Phase 2—State Route 26/Wabash Avenue

The "State Route 26/Wabash River Bridge" phase of the project involved the construction of two highway bridges—one at Columbia Street and one at South Street. These bridges established an overhead crossing of the future rail corridor, and due to the corridor's close proximity to the Wabash River, a new crossing of the river.

Upon completion of these bridges, the Main Street bridge over the river was closed to vehicular use. Construction was begun in 1991 and completed in 1992.

To facilitate alignment geometry constraints and clearance requirements over the future rail corridor, each of the 1,200 foot long bridges employed a "post-tensioned, cast in-place, variable depth, concrete box girder bridge" design.

Due to horizontal and vertical geometry constraints, a shallow deck section had to be employed over the corridor to achieve an acceptable vertical clearance relative to top of rail of the future tracks.

Phase 3—Ninth Street Underpass

The "Ninth Street Underpass" phase of the project was the first phase of construction in which CSX Transportation operations were impacted. This involved the relocation of one mile of CSX main and passing tracks, construction of a double-track bridge over Ninth Street, and the removal and reconstruction of twelve yard tracks within Lafayette Yard.

As a result of completion of this phase, one grade crossing was eliminated. Construction was begun in 1991 and completed in 1994.

Construction first began with the closure of Ninth Street and construction on the underpass.

Each girder is 113 feet long and 13' 3¼" deep (see Figure 3).

Since the use of the yard was critical from a train operating standpoint, removal and reconstruction of the yard had to be done in phases to minimize its impact on train movements.

Tracks were removed on the west half of the yard, leaving the east half in-service. Filling of the area commenced immediately thereafter using borrow from the underpass excavations and outside sources.

Considerable drainage improvements were included in the project. The reconstructed Lafayette Yard had 12-inch diameter, fully coated, perforated metal pipes installed to facilitate drainage.

In addition to the rail and highway improvements, reconfiguration and installation of numerous public utilities was necessary.

Several retaining walls were required as part of the relocation project since the new corridor passes through an urban area with limited room.

After completion of the grading, drainage and subballast for the west half of the yard, four yard tracks plus a portion of a fifth were constructed on the new grade. The fifth was constructed on a temporary alignment as a mainline "shoo-fly" or by-pass track for all through trains. This temporary main track allowed the removal of the old main track and the four yard tracks in the east half of the yard.

All tracks in the yard were of 112# relay welded rail on new timber crossties using conventional fastenings. Turnouts were no. 10-115#, hand operated.

The tracks were all surfaced using limestone ballast.

Figure 2 shows the yard tracks completed in the west half of the yard, and the nearly finished grade and subballast for the east half of the yard. The track on the left is the newly constructed yard track serving as the main track "shoo-fly."

After completion of the grading, drainage and subballast for the east half of the yard, five yard tracks plus the main track were constructed. The temporary "shoo-fly" track was realigned to its permanent location to form a yard track.

Figure 2. Completed Track in Lafayette Yard.

Figure 3. Completed Ninth Street Underpass.

Phase 4—CSX Relocation

The "CSX Relocation" segment of the project allowed the removal of trains from the center of Fifth Street, the primary goal of the project for CSX Transportation.

Work under this phase involved the relocation of 3 miles of CSX main track including two control points. To accommodate the relocation, it was necessary to erect one single-track bridge on abutments constructed under Phase 1; three overhead highway bridges; roadbed, drainage and utility alterations for the new CSX main track and the two future Norfolk Southern main tracks through the downtown area; and construction of the Depot Plaza, which will be a combined Amtrak/bus station for the city of Lafayette.

As a result of completion of this phase, CSX Transportation's main track was removed from 14 city blocks.

Construction on Phase 4 was begun in 1992 and was completed with the opening of the Depot Plaza in the summer of 1995.

The corridor is designed for a maximum authorized speed of 50 MPH. The maximum curvature on the main track is 3 degrees-30 minutes and the maximum grade is 0.75% (see Figure 4).

As grading in the corridor progressed, a portion of a timber wall of the former Wabash & Erie Canal was unexpectedly excavated. Due to the historical significance of this find, grading work was suspended to allow the site to be thoroughly documented by historians.

Although the northern half of the project had fairly good subgrade materials to work with, the southern half was found to be in poor condition with poor drainage. To help correct this condition, geogrid was used liberally to provide additional strength to the subgrade.

Figure 4. Side View of CSX Wabash Avenue Bridge—The bridge has a reinforced concrete ballast deck. Due to the curved intersecting alignments between the bridge and Wabash Avenue, the west girder is 87 feet long while the east girder is 89 feet long. Both are 9' 4" deep.

Figure 5. Completed track at Harrison Street. This is a view looking north with the Harrison Street overhead bridges in the foreground.

Figure 6. AMTRAK at Temporary Station. This is a view of Amtrak's "Cardinal" in the new corridor and of the shallow deck section for the overhead highway bridge.

The track was constructed of 122# relay welded rail on new timber crossties using conventional fastenings (see Figures 5 and 6).

The last day of operations down Fifth Street for CSX and Amtrak trains was on July 22, 1994. (The last freight train is shown on the cover of this Bulletin).

City-wide celebrations marked the passing of the last trains on the morning of the 22nd, and greeted the first train that evening after CSX track forces lined the mainline track to the new corridor track.

Depot Plaza

When operating down Fifth Street, Amtrak trains stopped at a store front for passengers. With the reroute of CSX and Amtrak trains to the new corridor, a new passenger station had to be provided. To fulfill this need, the city of Lafayette moved the former Big 4 depot from its location along the old Muncie Branch to a location on the new rail corridor.

The original depot is being renovated to create the second floor of a new 2-story building to be used as an Amtrak and mass transit station.

Since Norfolk Southern's double-track mainline will be located between the city's Depot Plaza and CSX Transportation's main track on which Amtrak operates, a pedestrian overhead bridge is currently under construction.

Phase 5—Norfolk Southern Relocation

The "Norfolk Southern Relocation" segment will be the final phase of the overall Lafayette Railroad Relocation Project.

Work under this phase will involve the relocation of 4.2 miles of Norfolk Southern's double-track mainline into the new rail corridor with CSX Transportation's main track. As part of this phase, a 150-car CSX/Norfolk Southern interchange yard will be constructed.

This phase will eliminate 24 grade crossings.

The proposed interchange yard will be located between the mainlines of CSX and Norfolk Southern. Both railroads will have direct connections to the yard at each end.

Completion of Phase 5 improvements is to be done in segments as funding becomes available. Work recently began on the CSX interchange lead bridge and the Norfolk Southern mainline bridge over Ninth Street.

The reroute of Norfolk Southern's trains to the new corridor is projected for 1997 and will result in the permanent closure of a total of forty-two at-grade street crossings.

FRA (FEDERAL RAILROAD ADMINISTRATION) WAIVER EXPERIENCE PANEL

(1) ATSF'S WAIVER FOR ELECTRONIC REPORTING

By: W. E. Russell*

To begin this panel discussion I will give you a little background about the reasons Santa Fe requested a waiver from the FRA with respect to electronic inspection records for its Signal and Track Teams.

Starting in 1986, Santa Fe, like many properties, started a major restructuring of its operations. Since 1986 to the present, we have gone through some eight different changes in regards to operations and administrative procedures. In 1986 we had 12 operating divisions, three grand divisions and a system office. Each of the 12 divisions, grand divisions and the system office had full engineering clerical staffs with anywhere from 10 to 15 clerks, backed up with a division filing system which was backed up with similar clerical and file systems at the grand division level and system level. Each division had just a little different way of doing things. As we began to down-size, close division offices and relocate administrative functions, our records did not always end up in the right place. Records were similar to the tools in my garage and basement, I know I've got them, I just don't know where they are located.

A FRA inspector might show up in Amarillo, Texas to inspect records, only to find out that the records requested were located in Albuquerque, New Mexico. The next year he would show up in Albuquerque and find out that the same records were now in Kansas city. You can play that game for only so long before the FRA inspectors start writing Code One's (i.e., citations for rule violations). There is nothing like the possible violation of a federal law to get you to thinking of a better way to do something.

We knew that our manual record keeping system had outlived its time and that electronic data entry systems are a way of life today and for the foreseeable future, so why not present our case to the FRA and develop a system that would satisfy the federal regulations as well as our own needs.

Our first approach was to visit with FRA officials, present the concept and get their input and concerns. In principle, they were receptive, but felt that a legal review by counsel would be best before jumping in with both feet. As with most issues that require legal opinion, you almost forget what the issues were when you first started out. Finally, the lawyers reached a meeting of the minds, which allowed both FRA and ourselves the freedom to come up with solutions that both satisfied the FRA's concerns and met our goals.

The FRA understandably wanted to ensure that no aspect of the reporting process would suffer with respect to timeliness, accuracy and security.

Both FRA and ourselves agreed that all electronic data entry would have to be completed within 24 hours of the physical inspection. This would accommodate our railroad inspectors, who might have to respond to an emergency situation and would not be able to download or enter the inspection data immediately following the inspection.

With regard to accuracy, once the inspector's information is transmitted to the mainframe computer, the report cannot be edited or changed in any respect. If there was a mistake made, such as a

*General Director, Roadway Maintenance, ATSF Ry

typo-error, another report would have to be entered amending the incorrect report. The amended report will have an explanation for the amendment. When training our inspectors, we are emphasizing the importance of proofreading their reports before transmittal. It won't be as easy as using the eraser on a number two pencil.

The FRA was concerned about the issue of electronic signatures. Who would have access to this information, who would be able to enter information, could anyone other than the inspector make changes to the reports? All of which were very good questions. The answers are very simple. Just as with your ATM (automatic teller machine) bank card, the electronic signature is only known by yourself and your bank. In this particular situation, the bank is the railroad's information systems. There is no list of the electronic signatures. Supervisors of the railroad inspectors can read and or print reports, but cannot alter the report in any fashion. We are of the opinion that this is a much more secure system than the present paper and punch system.

Where are we today in this project? The programming is completed for both the Signal Team Reports and the Track Reports. The Signal Team will begin its test area this month (i.e., March 1995), and the Track Team will begin its test in May. The difference in time is primarily due to the placement of a hardware, as we are working on a number of other electronic data entry projects for our track teams during this same time period.

Anyone who is interested in reviewing this project is welcome to come to Kansas City and visit with our Signal and Track Teams.

In summary, both the FRA and the Santa Fe see this type of reporting system as a very positive change and a major step forward that will bring cost benefits to both the FRA and the Rail Carrier, improve our flexibility, and make a FRA Inspector's job a little easier.

(2) BN'S WAIVER FOR THE USE OF THE BULLDOG CLAMP

By: M. N. Armstrong*

Thank you for the opportunity to discuss Burlington Northern's experience with the FRA waiver allowing us to use what we refer to as the "Bulldog Clamp." Since a picture is always worth a thousand words, I'll share some slides of the clamping device with you as I go through this presentation. To start off with; however, I'd like to recognize FRA employee Russ Bunker. Russ was very helpful during the initial stages of the waiver application and we appreciate his assistance and expertise very much.

Why did Burlington Northern consider utilizing the Bulldog Clamp as an alternative method for securing rail defects prior to their repair? First, I think it is important to point out that we requested to use the clamps on our heavy haul coal territory, where we operate a rail detector approximately every 35 days on single track. In these locations, vertical head loss of rail can be as high as 5/8" on 136# rail which, at times, makes it difficult to match a cut-in plug rail. Secondly, our initial plans were to make repairs to some of the defects using wide gap welds. A lesson we have learned in the heavy haul territories is that you need to eliminate any and all bolt holes that you can, since cracks tend to propagate from the holes, which will ultimately lead to rail failure. Fortunately, certain rail defects protected by the Bulldog Clamp allow for a thermite weld repair to be accomplished without bolt holes remaining in the track.

The Bulldog Clamp is a system consisting of two unpunched bars manufactured to A.R.E.A. (American Railway Engineering Association) cross-sectional and metallurgical specifications, along

*Assistant Chief Engineer, Maintenance Planning, Burlington Northern

with two large clamps to hold the bars in position within the web of the rail. The bars have been designed to accommodate the thicker section in the web area that is encountered with today's thermite field welds. The clamps are cut from 2" thick, hot rolled plate steel, A36 specification. The clamping pressure is applied to the bars through the torquing of one 1" A490 specification structural bolt per clamp. The bars are placed against the web of the rail and centered around the field weld or other defect. A clamp is then placed on both sides of the defect. The bars are designed to support the head of the rail and add stiffness over the 20" on either side of the defect, while not actually contacting the defect. Allegheny Rail Products is the manufacturer of the clamp that we are currently using (see Figures 1, 2, 3, and 4).

The two year waiver, which BN was granted by FRA on July 26, 1991, contained, in part, the following conditions:

1. Once the clamps are applied to detail fractures, engine burn fractures or defective welds measuring 20% or greater of head area, speed shall be limited to 45 MPH or the maximum for the class, whichever is lower.
2. The clamp shall be removed not longer than 5 days after application. If the internal defect has not been removed in that time, bolted joint bars will be immediately applied.
3. The waiver does not apply to damaged rail.
4. BN shall maintain, in a central location, the following information pertaining to each application of the clamp:
   a. Location of clamp application
   b. Date of clamp application
   c. Date of removal of clamp
   d. If a service break occurred with the clamp, length of pull-apart and ambient temperature.
5. BN shall send a monthly report to FRA containing the information stated previously. The report must be submitted within 30 days after the expiration of the month during which the clamp was removed from rail.
6. BN shall notify the FRA immediately in the event of a derailment involving rail on which a clamp has been applied.

The 5 day limit regarding removal of the clamp was somewhat difficult to manage and we were, therefore, fairly conservative as to how we were going to responsibly manage this effort. As a result, we have limited the use of the Bulldog Clamp to one Roadmaster's territory between Edgemont, South Dakota and Gillette, Wyoming. We hope that we are demonstrating responsible use of the clamp so that in the future consideration may be given toward a less restrictive limit on how long the clamps can remain applied to the defect. Ultimately, we would like to have 20 days in which to remove the clamp and perform a repair.

Near the expiration of the original two year waiver we, again, approached the FRA asking for an extension of the waiver for another 24 months, and to raise the 5 day limit to 20 days. The FRA did grant an extension to the waiver on June 21, 1994; however, the limit was only raised from 5 days to 10 days. The 10 day limit has helped us in the management of the clamps, although, once again we hope that in the future a 20 day threshold can be achieved.

Currently we are repairing some Detail Fracture type defects with a 1" thermite weld. Our original plans to use 2" and 3" wide gap welds for defect repair have been altered since, at this point in time, the wide gap welds do not provide the service reliability which we require. We hope that in the not too distant future the wide gap welds can be improved so that they do perform reliably in-tract and provide a much needed alternative to defective weld repair.

The defect protection and repair process for detail fractures (i.e., DF) begins with the rail detector car operator accurately marking the location of the DF. A Bulldog Clamp is then applied to the defect until the thermite welders can get into position to make the weld. Applying the Bulldog Clamp consists of removing the ballast to accommodate the clamps, followed by placement of the bars and clamps in the prescribed manner. A standard track wrench is used to apply the same torque to the locking bolt and nut that would be applied to a standard track bolt.

Figure 1

Figure 2

Figure 3

Figure 4

When the welders move into position, they cut the rail 1/2" each side of the marked DF to provide the minimum 1" gap that is required for making a thermite weld. We believe that this repair method has merit, since it eliminates the DF, does not require matching of a plug rail and is accomplished with one thermite weld instead of two. Overall, the quality of the track structure is enhanced and the time to make the repair is reduced.

In summary, we are attempting to accomplish the following:

WHAT ARE WE DOING?

—Applying a clamp to the verified rail defect.

—Providing protection for the defect without adding bolt holes to the rail.

WHY ARE WE DOING IT?

—To provide protection to defective welds until a permanent plug rail repair can be made.

—In the event that we can repair a DF with a 1" thermite weld, we avoid handling of the plug rail and the resulting plug rail mismatch.

WHERE ARE WE DOING IT?

—Heavy haul coal routes.

—At the present time, however, we are only using the clamps and repair process between Edgemont, South Dakota and Gillette, Wyoming.

Current plans call for us to monitor our satisfaction and confidence in the thermite weld repair process that is used in conjunction with the Bulldog Clamps and detail fractures. At the expiration of this current waiver, we would like consideration to be given to raising of the time limit for removal of the Bulldog Clamp from 10 days to 20 days. We also hope at some point in the future to be able to use the Bulldog Clamp and weld repair process across our entire system. As a final note, Burlington Northern appreciates the FRA's review, consideration and granting of our waiver application.

Thank you very much for your attention.

(3) CSXT'S WAIVER FOR USE OF GRMS SPLIT AXLE TEST VEHICLE

By: J. C. Tomkins*

Gage Restraint Measurement System (GRMS)

GRMS is a performance based track strength evaluation system, designed to improve railroad safety and maintenance efficiency. The test equipment provides the accurate measurement of the gage holding ability of ties and fasteners at test speeds up to 30 mph. The "split-axle" device applies a controlled non-destructive gage widening load to the track structure. Locations exceeding defined parameters are precisely identified and field marked for inspection and correction as necessary. Computer monitored testing provides a data base that can be used for maintenance planning, track degradation study and line segment comparisons. In summary, the GRMS provides an objective analysis of track strength and eliminates the subjectivity of visual inspection.

Waiver Purpose

To demonstrate the feasibility of a performance based track safety standard relative to gage.

Waiver Experience

This demonstration project and the FRA waiver were fully implemented on 500 miles of CSXT's Florence Division in 1994. Concerns regarding waiver prescribed remedial actions as a result of split-axle testing have been discussed openly with the FRA and modified to suit the objectives of both the FRA and CSXT. One such concern was the waiver definition of test exceptions

*Division Engineer, CSX Transportation

requiring the application of 10 mph slow orders behind the test vehicle. Modification of the exception parameters dramatically reduced the slow orders generated by the test vehicle with no compromise in safety.

As we gain project experience, we fully anticipate further modifications to the waiver as we move away from technical specification remedial action.

GRMS testing could not have been fully implemented in the absence of a FRA waiver. This waiver was developed in a spirit of cooperation and put in place with a minimum of difficulty. It has thus far proven beneficial to both the Federal Railroad Administration and CSXT.

Waiver Benefits

- Provide a regulatory environment supportive of new technology testing and advancement.
- Provide an objective technical analysis of track structure integrity.
- Provide positive and productive regulatory enforcement.
- Provide enhanced operational safety.

(4) UP'S DETECTOR CAR WAIVER PETITION

By: J. M. Haverstick*

Original Rail Testing Plan

- This concept is for improving railroad safety on UPRR.

Goals

- Improve detector car mileage and cyclic times.
- Locate and remove critical defects.
- Reduce service failures.

Why

- Increases in management make it more critical to improve test cycles.
- Increases in rail life have the potential to increase the number of defects.

Currently (1992)

- Seven detector cars and one supplemental car are used annually.
- Currently tested 35,219 miles annually or 21.70 miles per day.

Proposal (1993)

- Allow detector car to provide electronic data to chase car.
- Seven detector cars full time and one supplement.
- Increase to 50,000 miles annually or 31 miles per day.
- All (critical) indications verified and required action performed immediately. *Critical Defects*—All fissures, welds and engine burns—GT (greater than) 25%, HSH (horizontal split head),

*Director, Track Test & Track Weld, Union Pacific R.R.

SW (split web) and BB (broken base)—GT 6", VSH (vertical split head) and PR—GT 6", HW (head and web)—GT 3" and BH's (bolt hole cracks)—GT 1" in jointed track only.

- Chase vehicle, gang or in-track welder to verify (non-critical) indications.
- All (non-critical) indications would be verified and removed within three days.
- Basic concept is much the same as currently in use on the BNRR.
- Implement plan system-wide.

Note: All verified non-critical defects left in track exceeding three day grace period would be subject to FRA standards.

Benefits

- Improved safety by increased test miles and decreased cycle times.
- Crews will remove more defects by concentrating efforts where needed.
- Reduced derailment threat by reducing service failures.

Approved Rail Testing Plan

Approved April 20 (1994)

- Implement plan as a test between North Platte, Nebraska and Point of Rocks, Wyoming.
- Implement plan for one year.
- Detector car to verify all defects by hand-test within two hours of identification.
- This test plan covers one detector car's territory.
- All (critical) indications verified and required action performed immediately. *Critical Defects*—All fissures, welds and engine burns—GT 15%, HSH, SW and BB—GT 6", VSH and PR—GT 6", HW—GT 3" and BH's—GT 1/2" in jointed track only.
- A three day grace period is granted on all (non-critical) defects. Defects left in longer will be restricted to a 10 MPH slow order.

Note: When temperature drops below 0 degrees Fahrenheit all fissures, welds and engine burns—GT 10% are classified as critical.

Conclusion

- A 21% decrease in mileage has occurred 1994 vs. 1993 for this test car. This is primarily due to a reduction in track time.
- The plan has not been an overall success due to two factors: 1) track time dwindling, and 2) field forces not fond of leaving any defects to correct later.
- The FRA worked closely with us to make this plan a success.
- We were disappointed in two areas: 1) time to get approval, and 2) additional restrictions.

Where Do We Go From Here?

- A task team has been assembled, consisting of Field and Harriman Dispatching directors and managers and our Detector Car group, to address this question.

MBTA'S RED LINE—DIRECT RAIL FIXATION FASTENING SYSTEM REPLACEMENT

By: Robert Johnson*

My talk is about track construction, not just any type of track construction, but the replacement of a failing direct rail fixation system on two transit bridge structures.

The Savin Hill Flyover and Anderson Bridge were constructed in 1969 and opened for revenue service in 1972. They are the two primary structures allowing MBTA Red Line Rapid Transit trains to travel from south of Boston to downtown Boston. The structures are simple span in nature, constructed of concrete with steel spans, a two part concrete deck consisting of a deck slab and a plinth that supports the rail system. The length of direct rail fixation for both structures is approximately 4,200 lineal feet.

The rail section for these structures is 115# continuous welded rail, which lays on top of a rubber monolithic boot that sits on a transite pad embedded into the concrete plinth. The rail is secured to the concrete plinth with a Moses clip. Third rail sections are 150# supported by fiberglass insulators fastened to smaller plinth sections which are parallel to the running rail.

In 1975, the maintenance of way department began to notice what appeared to be surface cracking in areas around some of the Moses clips and the sliding joints which were located on the bridge structures. Track walkers monitored the situation until failure of the horizontal alignment seemed eminent. Remedial efforts to repair the concrete were attempted, including patching the concrete and introducing a new fastening system. These methods failed primarily due to poor installation by the inexperienced maintenance forces. All temporary remedies were soon proven ineffective. During the mid to late '80s it became apparent that the ongoing maintenance effort was as effective as the dutch boy attempting to plug up the leaky dike with his fingers. As each temporary repair was attempted, a new failed section would spring up causing concerns for the integrity of the railroad.

With all of the interim repairs failing, the most dreaded of actions to an operating rail road was implemented, a speed restriction. The speed over both structures was reduced from 50 to 25 mph and with a two track transit system, you can well imagine the effect this had for the MBTA's Red Line service. Reducing the speed from 50 mph to 25 mph removed the word *rapid* from *rapid* transit.

It must be said that there is no one villain for this failure. As it sometimes happens in our business, we seek new technology that, when effective, can save thousands of dollars in maintenance costs. In some instances the leading edge technology that has been carefully scrutinized passes all of our tests, but fails the test of time. Failed transite pads combined with opposing thermal stresses, loss resiliency for the rubber pads, toe loaded clips and the poor location of the sliding joints all must take part of the blame. Studies have shown that the installation or continuous welded rail and sliding joints on the concrete bridge deck have caused a conflict during the thermal expansion and contraction of the CWR (continuous welded rail), and the bridge structure.

With a toe loaded fastening system in place and with different expansion ratios, unanticipated stresses occurred in the areas around the sliding joints and Moses clips, causing distress and deterioration of the concrete. This, combined with the freeze thaw cycle of the winter, helped to exacerbate the situation.

With the speed restrictions in place and the condition on the bridge degenerating, the MBTA was faced with a major dilemma. How could this situation be remedied without affecting revenue service?

*Project Manager, Massachusetts Bay Transportation Authority (MBTA)

After several in house meetings, it was determined that the Engineering and Maintenance Department had exhausted its chances to make reliable interim repairs, and that a permanent solution would have to be achieved. At this point, the construction directorate was brought in for help.

The first course of action the Construction Directorate undertook was to have an engineering firm evaluate the problem and provide the MBTA with recommendations. We needed to know how permanent repairs could be made, while maintaining the main artery of transit service to the south shore of Boston.

Fortunately, the project already had a design team on board evaluating the state of the Red and Orange Line's rights of way. This saved critical time that would be lost during the R.R.P. (request for proposal) process. Immediately, the design priority shifted to the failing fastening system and how to fix it. At first glance we had to establish what type of repairs were required, and basically it came down to three. One would be a patch method, another would be converting the bridge structure to a ballasted deck type, and the third would require complete demolition and replacement of the failing concrete and the installation of a new type of rail fastening system. With the modifications learned from the original design, the last thing we needed was for history to repeat itself. Therefore, our solution was to move the sliding joints off the bridge, to eliminate thermal conflicts and install Cologne Egg fasteners using a no toe load Pandrol clip, allowing the rail to expand and contract independent of the bridge structure. Our goal was now established, to eliminate the decaying plinth, install a new direct fastening system and to do it without major interruption of service. A Herculean task to say the least.

With the proposed remedy in hand, the next objective was to establish an inter-departmental task force to provide the project guidance, not only from an operational and maintenance perspective, but also to advise the project on how to interface construction with operations and have the least impact to our customers and the surrounding community.

As it turned out, the easy part was the corrective measures. The difficult part was how to chip out failed concrete, remove the old fastening system and install a new fastening system. Even this part would have been relatively easy with a track outage. The tough question was how to do it with the least impact on our revenue service.

The designer had a lot of tough questions to answer and engineering to provide, but the critical question that remained was how to make repairs without totally shutting down service. It must be noted that weekday busing of this line was impossible due to traffic restrictions.

The consultant and project team came up with several options, including truncating the South Shore tracks, then tying them into the Red Line's Dorchester branch which ran parallel to the South Shore tracks in one location. This option would require busing three stations on the Dorchester branch and would only benefit the Savin Hill Flyover. From the project's perspective this was a dream come true, but from the perspective of the Dorchester community (a precinct of Boston) came a political backlash. "Why are we being inconvenienced for the South Shore ridership?"

Good question—back to the drawing board. Another thought was to single track the South Shore branch allowing construction on one track and revenue service on the other. Unfortunately, head way analysis on our single track transit system proved that this would be a commuter nightmare, rippling service delays as far north as Harvard Square.

With time running out and no great solution in hand the next construction season seemed in jeopardy.

It was at this time that lead engineer Dominic Bua of GB&R (Gordon, Bua & Read, Inc.) proposed an innovative solution (at least by MBTA standards), to build a panelized temporary bridge structure over the existing tracks. This would enable trains to run weekdays over the temporary bridge while maintaining the restricted speed of 25 mph. Utilizing weekend shut downs, temporary bridging panels could be removed and repairs could be made. This system provided for the resump-

tion of revenue service on Monday mornings as well as allowing for proper curing time for the freshly placed concrete. Using this procedure we were able to achieve the desired quality of a complete shut down of service.

Now that we had a construction solution and an apparent operational solution, we still had to sell the bridge over the bridge idea to our Operations Directorate, and we all know how flexible operations can be.

The temporary bridging proposal seemed to solve all of the construction requirements, but to convince the Operations Directorate that it could work was a difficult task. We had to rely on the Engineering and Maintenance Department and Construction's own track and design guru, John I. Williams and then Assistant General Manager Lyn Wylder, to help persuade the skeptics in subway operations. Once Operations was convinced, the designer was given the green light and proceeded to finalize the design so that the MBTA could advertise a construction contract.

While the contract was being advertised, a two-fold public relations campaign was underway. The first was to acquaint our weekend Red Line riders with the busing diversion established by our Operations Department. In order to effectively reach our ridership, brochures explaining the project and its ramifications on service were distributed at all Red Line South Shore stations. Project posters were displayed within the stations and carcards were installed on the Red Line cars. To further inform the riding public, on the advent of any three day weekend work window in which a busing diversion was implemented Friday evening, notifications were placed in local newspapers as well as flyers distributed to the public by the operating personnel.

The second portion of our public relations campaign was to inform our abutters. This was accomplished by project personnel attending community meetings to explain the scope and schedule of the project, and of course distributing flyers throughout the affected neighborhoods prior to around the clock weekend work.

Internally, there was a significant amount of coordination needed between Engineering and Maintenance's (E&M) forces, Operation officials and the Construction Directorate. In addition to the contractor/MBTA joint installation of two universal crossovers for emergency measures, E&M forces were responsible for de-energizing the third rail and grounding it to the running rail, and installing and removing chained ties and lanterns and portable trips at the job sites' perimeters at the beginning and end of each weekend.

The Operations Directorate was also heavily involved in the day-to-day activities on this project. From an administrative view point, Operations assisted the project in meeting their budget constraints by coordinating their busing picks within the construction schedule, thus eliminating costly overtime. Operations also assigned transportation officials to assist with the off-loading of passengers within the diversion area each weekend. And most importantly, transportation officials participated in the time consuming test train procedure performed at the end of each work window prior to turning the tracks over for revenue service.

Within the first few weekends of construction, all of this interdepartmental coordination had established a rhythm which had a profoundly positive effect on the construction progress of this project.

Figure 1. This is a birds eye view of the Savin Hill Flyover. As previously stated a concrete slab bridge deck with second pour concrete plinth. The running rail is fastened to the second pour with Moses clips. From this perspective you must be asking what's the big deal.

Figure 2. Note some of the concrete degradation around the Moses clips. The lighter squared sections around the Moses clips are the transite pads embedded in concrete now being exposed due to the pumping action created by the fastening failure.

Figure 3. Note some failed temporary patching performed around the Moses clip as well as an attempt at permanent repair. A Pandrol anchoring insert with a Pandrol clip was installed sporadically to act in concert with the Moses clips. This method of repair proved ineffective due to poor installation and because of the pumping action of the rail, which allowed the Pandrol clip to vibrate free.

Figure 4. A closer view illustrates how the pumping action of the rail literally scored its way into the plinth, thus rendering the temporary repair ineffective.

Figure 5. This area is indicative of the condition at its worst. Gauging rods were used in all areas similar to this, but this photo clearly gives you an understanding of how critical the problem was.

Figure 6. Not only did this condition weaken the track structure, but the emergency measures taken affected the integrity of the bridge guard. Torch cutting of the bridge guard was required to accommodate the installation of gauging rods.

Figure 7. Note the two Moses clips with the concrete failing around them. The upper clip is a good representation of the transite pads that were abated. Fortunately the abatement procedure went smoothly and most of the pads came out in one piece.

Figure 8. With failing concrete, this condition was a maintenance nightmare.

Figure 9. This scene gives an interesting perspective to this project. In the lower portion are two attempts at patching the Moses clips; further down, note the sliding joints located on the bridge deck.

Figure 10. This was a test pit taken to examine the rebar. For the most part it was in decent shape. Again, note the transite pads being exposed.

Figure 11. Sliding joints located on the bridge deck combined with a toe loaded fastening system and failed transite pads are suspected to be a formula for disaster.

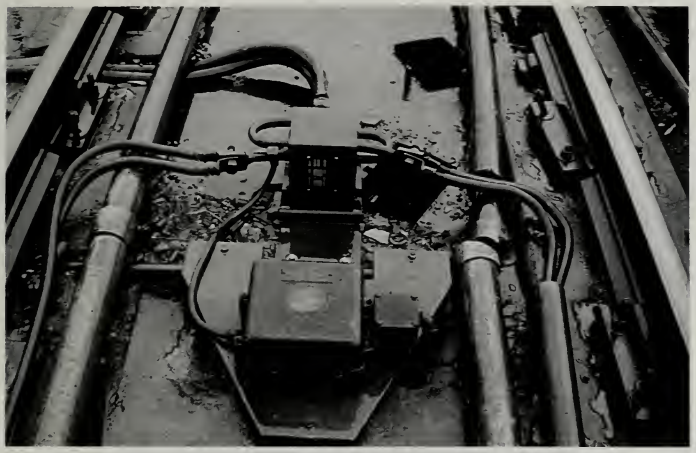

Figure 12. If repairing the failing fastening system on weekends wasn't enough, it was necessary to factor in the protection of the sensitive Wee Zee bonds. Wooden protective boxes were installed weekly to aide in the protection.

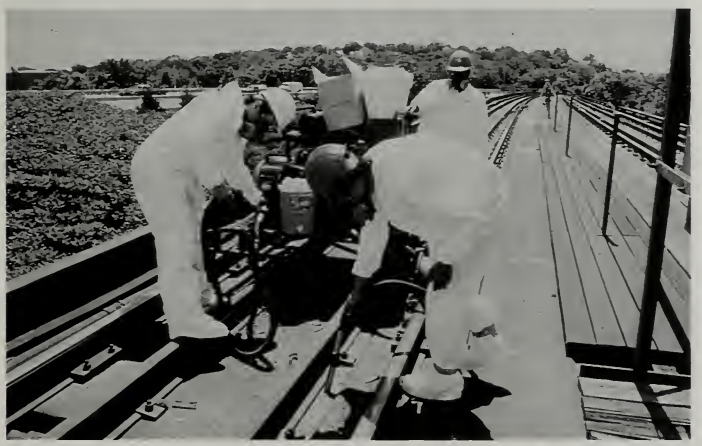

Figure 13. The first operation to be performed was the pre-cleaning of both bridge decks, removing any loose sections of the transite pads.

Figure 14. After the bridges were pre-cleaned and prior to any concrete demolition, the transite pads were abated. Note the Tyvek suited abatement workers in the foreground.

Figure 15. Prior to any abatement or construction, the contractor constructed the temporary bridge deck. Basically, this had timber beams anchored to the bridge deck parallel to the running rails—one beam on the field side, two inside the running rails and one on the dummy side. The timber beams were secured by drilling anchor holes into the bridge deck, applying epoxy into the drilled hole and installing a Hilty fastening system. Angle irons for the deck beams were used on the Savin Hill Flyover only to supply additional support for the temporary bridge in areas of superelevation. In this area, there was up to 6 inches of superelevation.

Figure 16. Pre-plated wooden timbers similar to cross ties supported with reinforcing angle iron were then placed horizontally across the beams and then lagged into the beams.

Figure 17. Running rail was then installed using a Pandrol fastening system. This prevented the spike killing of the wooden timbers due to the constant removal of the running rail during construction. The rail length varied, but was mostly 19'6" for easy handling in panel form.

Figure 18. Tie bars were used to prevent the shifting of the wooden timbers, similar to what is used for a timber trestle bridge.

Figure 19. The temporary bridge provided the MBTA with the desired flexibility to run revenue service and make the necessary repairs to the failing fastening system.

Figure 20. To handle the transition from the bridge deck to the existing track each timber was milled to a different height, gradually reducing the thickness from the apex allowing for a smooth run off to the existing track. This allowed the contractor to modestly raise the existing track to meet the new profile of the temporary bridge deck minimizing a very difficult under cutting operation when the temporary deck was removed.

Figure 21. A profile view shows the angle iron support resting on the timbers.

Figure 22. This shot is typical of the activity underway during the demolition stage. Where the compressor is sitting are sections of the temporary bridge that were not removed, and work crews demolishing the degraded concrete. To the right of the crews is one track nearly completed with the new Egg fasteners and composite third rail installed.

Figure 23. Demolition crews on the Savin Hill Flyover illustrate just how labor intensive this project was.

Figure 24. The bottom left portion of this picture illustrates the scored concrete from the rail after the fastening system had failed, and the relatively good rebar exposed by the demolition.

Figure 25. Because one track was always maintained for high rail access, demolished concrete was removed in small dumpsters that were placed within the work area.

Figure 26. Dumpsters were lifted and placed on a flatbed, and brought to a staging area for further handling. This flatbed system was also used to remove the temporary bridge panels from the work site.

Figure 27. After concrete was removed, the area was swept clean, acid washed, rebar wire brushed and epoxy coated and the entire trough was sprayed with a bonding agent (Sika Armatec 110).

Figure 28. A worker inspects the area where rebar repair will be needed.

Figure 29. With form work installed and rebar repaired, the contractor was ready to place concrete.

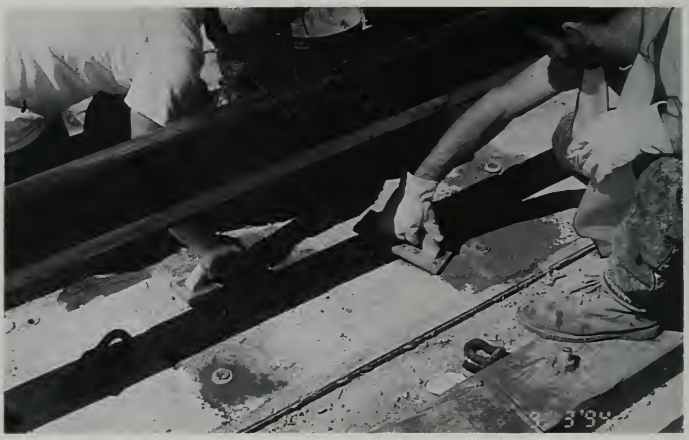

Figure 30. Anchor bolts for the new fastening system were affixed to a laminated template that was supported by plinth form. Concrete was delivered by 1 and 2 yard hoppers brought to the work area by high rail cranes. Due to the restricted work window, a 5,000 PSI high early concrete was used to accelerate the curing process. Placed concrete was allowed to cure to a point where the templates could be removed and concrete around anchors could be worked.

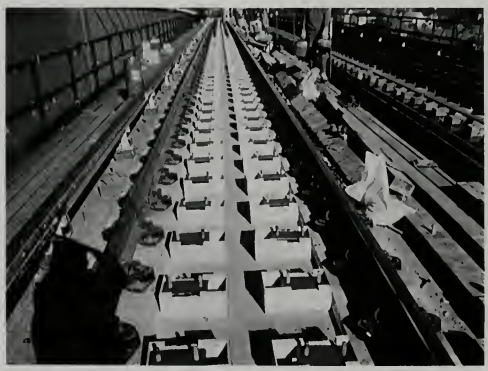

Figure 31. Concrete was placed at various intervals from 300 to 600 linear feet. Once the placement effort was complete and concrete properly cured, the temporary bridge was removed, Egg fasteners were set and the rail was installed. With new fasteners in place the contractor then installed smaller plinths to support the bridge guard.

Figure 32. With the new Egg fastening system in place, and the bridge guard and CWR installed (not seen in this photo) the speed restriction was lifted from 25 mph and returned to the original design speed of 50 mph, providing MBTA customers with a smoother, faster and safer commute.

AN EVALUATION OF REMEDIATION TECHNIQUES FOR CONCRETE TIE RAIL SEAT ABRASION IN THE FAST ENVIRONMENT

By: Richard P. Reiff*

Background and Scope of Concrete Crosstie Rail Seat Abrasion

Concrete crosstie rail seat abrasion was first recognized as a problem in the late 1980's. An early industry survey revealed that 15% or more of concrete railroad ties in service for five years or more required some early repair. Closer analysis of this survey showed particular climates, track geometry and loading conditions where the abrasion was taking place. In 1993, railroads reported repairing 500,000 rail seats at a cost of $5,000,000. Additional costs incurred as a result of service disruptions were not included.

During the past four years, a large matrix of materials intended to eliminate or control concrete tie abrasion has been monitored as part of the Facility for Accelerated Service Testing (FAST)/Heavy Axle Load (HAL) program at AAR's Transportation Technology Center, Pueblo, Colorado. Results after 300 MGT (million gross tons) of HAL traffic point to new repair techniques as well as several promising preventative measures for resisting abrasion. Further testing of matrix materials will be needed to determine whether the most promising methods have a sufficient life cycle to be considered a permanent solution or whether they require periodic inspection and replacement to maintain abrasion resistance.

In addition to tests conducted at FAST, a number of field sites in revenue service contain segments where new materials are being evaluated. Results from these sites also should be evaluated to determine material performance.

The "life between material replacement" is targeted as a minimum measurement of the rail replacement life cycle for maintaining abrasion resistance. This life cycle could range from 500 to over 1000 MGT on curves which may be subject to abrasion or between three and fifteen years, depending on local conditions (rail type, curvature, traffic, axle loads, speeds, etc.) under present heavy haul operations.

What is Rail Seat Abrasion?

Concrete tie abrasion results when tie material under the pad is lost to abrasion, hydraulic action and motion transmitted from the rail. The loss appears in the form of a localized breakdown of cement paste under the rail seat. (See Figure 1.) Abrasion is typically compared, in appearance, to tie plate cutting in wood ties. Wood, however, is subject to both fiber crushing and decay, which allows the tie plate to cut into the surface of the tie.

Overview

Rail seat abrasion became a concern in the mid to late 1980's when several North American concrete tie users noted significant >0.08-inch (> 2-mm) abrasion depths at certain locations. Some railroads reported extreme cases of up to 1/2-inch depth before repair and/or remedial techniques were applied.

Abrasion was noted in a number of areas. Wherever significant, the following characteristics existed:

*Principal Investigator, Metallurgy and Civil Engineering, AAR Transportation Technology Center

Figure 1.

- High annual tonnage
- High moisture (rain, snow, etc.)
- Severe curvature and high L/V conditions
- Heavy axle loads
- Rail grinding
- Use of locomotive sanding
- Ties equipped with EVA pads

These characteristics are often found in the severe areas of heavy haul railroads where concrete ties had been installed to eliminate previous problems such as wood tie splitting, gage widening and tie plate cutting. With rail seat abrasion, the primary concern is that excessive loss of rail seat depth can reduce the effectiveness of conventional elastic fasteners, which can lead to fastener fallout, running rail and wide gage.

Locating Rail Seat Abrasion

Unless excessive, abrasion is generally hidden from view by normal track inspection techniques. The most reliable procedure to inspect, measure and quantify abrasion requires removal of rail and tie pads so that the rail seat area can be observed closely. In addition to monitoring locations listed above, the following signs and indications can determine where more detailed visual inspections are warranted.

Automated Inspection

Automated track inspection vehicles, such as track geometry cars (TGC) and track loading (gage restraint measurement devices) produce large data bases, parts of which can be used to indicate abrasion. These indications include:

- Increase in gage widening between successive TGC runs that cannot be explained by rail wear.
- Uniform change in superelevation between successive runs which may indicate significant low rail abrasion.
- Localized and unexplained wide gage in otherwise "good" track.
- Large differentials between loaded and unloaded track gage.

All of these indications could be the result of track problems in addition to abrasion. As such, they can be used as a flag for more detailed follow-up field inspection.

Manual/Visual Inspection

Once an area has been identified as a potential site for abrasion, closer inspection will be needed to verify whether it exists, and if so, the depth and extent. This inspection will help dictate the type of remedial action to take. Visual indicators include:

- Powder or residue leaching from beneath the tie pad area
- Relative relationship of tie pad surface to top of tie
- Tie pad edges which are turned up and curled
- Frequent clip fallout without apparent reason
- Longitudinal running of the rail due to reduction in clip hold-down force
- Wear on insulator components and, if any, on fastener shoulders
- Wide static gage
- Change in rail cant, often noted during profile grinding efforts

Once ties have been inspected and abrasion has been confirmed, a number of options exist. If the abrasion depth is minimal or non-existent, the existing pads may be acceptable for limited continued use. As an alternative, new pad materials that are able to inhibit additional abrasion should be considered. Abrasion depths which might result in loss of fastener hold-down should be repaired by installing materials that restore rail seat height and rail cant to original configurations. Repair materials should not only resist abrasion, but last the life of the tie.

Evaluation of Materials to Reduce Rail Seat Abrasion at FAST

In response to industry concerns about abrasion, a limited evaluation of methods to eliminate, reduce or repair abrasion was installed at FAST in 1991. The original objective was to "determine methods and material combinations, in-field repairs, and manufacturing process, that are effective in preventing, reducing, or stopping rail seat abrasion."

In order to address this objective, a number of new concrete ties and ties from revenue service with various amounts of abrasion were obtained and installed in FAST during the December, 1990 track rebuild. A range of tie pads and repair techniques was introduced into this matrix of concrete ties. During the initial and follow-up phases of the FAST abrasion test, the matrix of ties and pads installed was accomplished under the direction of the Ad-hoc Concrete Tie Rail Seat Abrasion Committee. Committee members included representatives from railroads, suppliers, and the academic and research communities.

Tests and evaluations conducted at FAST were selected by the committee based on projected performance of new technologies being offered, and results observed from various field locations. In addition to the tests at FAST, a number of materials and methods are being evaluated by railroads and the supply industry in revenue service. For example, ties manufactured with convex rail seats were first installed in one revenue service site. Initial examination of this site (after about 60 MGT of traffic) indicated promising results, and additional ties of this design have been provided for evaluation in FAST.

Field sites offer a wider range of climate and operating conditions than at the TTC, while FAST provides a known, controllable environment and the opportunity for precise, periodic inspection and measurement. Data and results for this presentation are based primarily on observations and measurements from materials located in the FAST track. Results from both FAST and field/revenue service evaluations should be examined before definite conclusions can be drawn.

Two test zones at FAST which surround the existing concrete tie test zone in Section 3 (a 5° curve) were selected. (See Figure 2.) The relatively sparse natural moisture normally available in the Pueblo area required supplemental tie watering once every second day of operation. After committee members inspected the test at 30 MGT, watering was increased to twice during each shift of train operation. Using a specially fitted truck to spray the additional water, approximately 21 inches of annual effective moisture was added to the natural 11 inches per year. Thus FAST abrasion zones simulate an environment where approximately 32 inches of rain falls on the track annually.

The matrix of test ties and pads has undergone two revisions, once after about 130 MGT of traffic and again after 200 MGT of traffic. These revisions allowed for installation of new materials not previously available, and eliminated certain materials that did not appear to exhibit any abrasion reduction capabilities.

A detailed list of the latest matrix is included in the attached appendix. Major combinations of pads and repair techniques included variations in the following categories:

Dual Durometer Rubber pads

Rubber/reinforced pads

EVA pads*

Various pad materials with various shape factors

Polyurethane

Tie pads bonded/glued to the tie surface

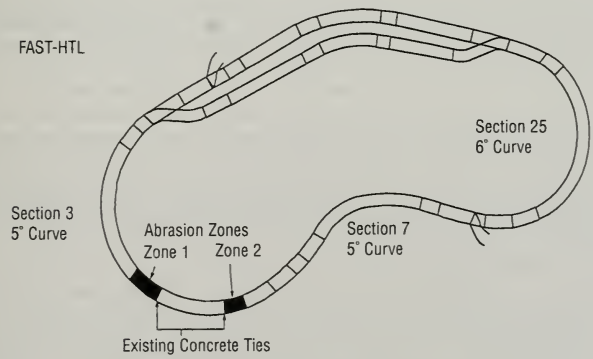

Figure 2. Concrete Tie Rail Seat Abrasion Experiment.

*No one has suggested EVA pads are a cure—they are part of the problem.

Pads with a sealing ring built into the surface

Three part sandwich pads (foam/steel plate/pad material)

Steel plate cast into new tie

Steel plate/epoxy repair

Epoxy repair

Variety of filler materials for repair

Alternative insulator materials

Tie surface coatings

Alternative concrete mixes

Rail seat surface shape factors

Note that EVA pads were included in the FAST test matrix as a "control," not as a proposed cure. Field sites equipped with EVA pads have often exhibited abrasion. These pads were included at FAST to determine the scope of abrasion under conditions at FAST.

The attached Tables 1 and 2 are a summary of the major differences in component materials. Each system offers a different approach to solving the abrasion problem. For example, glued pads sealed to the tie surface could eliminate infiltration of water, sand and other particles, thereby eliminating the sources of abrasion. Unfortunately, this particular matrix was not successful in reducing abrasion at FAST. The glue used was ineffective in adhering the pads to the tie surface for any duration (less than 2 MGT). Perhaps glue materials developed in the future will prove to be longer lasting.

The limitations of this abrasion test summary cannot accommodate both a complete description of all 65+ pad/material combinations and the theory behind each technique.

Data

Few data collection cycles were available to measure the abrasion depth, primarily because rail had to be removed, pads had to be individually picked up and the rail seat had to be cleaned. Data was initially obtained on a 50 MGT cycle; however, it became apparent that inspection and measurement processes could adversely affect the ability of some pad combinations to continue resisting the abrasion process. Subsequent measurements were conducted on 100+ MGT cycles.

A modified version of an existing abrasion gage utilizing a dial indicator was fabricated. This modification improved the data resolution of a tie pad footprint on the rail seat, which is measured with respect to depth relative to the non-wearing tie surface. Initially, the data was collected with mechanical dial indicator gages, however, updated versions now permit automated and continuous data collection. Nonetheless, this measurement is time consuming and conducted only at infrequent (50-100 MGT) intervals. A sample data plot showing two successive measurements is shown in Figure 3.

Results/Observations to Date

Has the rail seat abrasion problem been solved? Based on the solutions under evaluation at FAST, it appears that abrasion can be controlled by certain combinations of pads and materials, but the life cycle of these techniques has not been fully quantified. Several solutions including very soft pads, dual-durometer pads, and sandwich pads appear to either prevent or minimize the formation of abrasion. At this time, though, they do not appear to be permanent solutions; they must be replaced during the life of the tie or during rail replacement. Many of the softest rubber pads at FAST have shown no abrasion, but after 100 to 150 MGT of service, the pad has worn through or must otherwise be replaced.

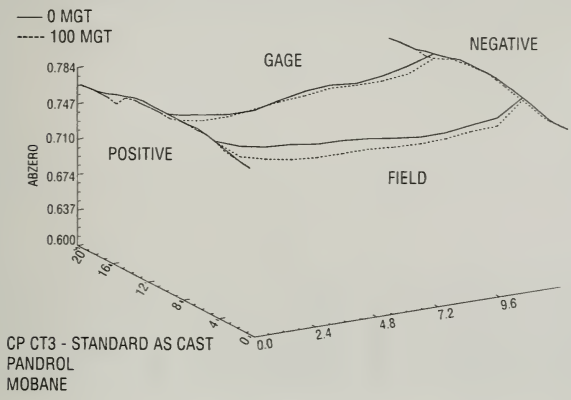

Figure 3. Average Depth Profile

If the life of abrasion eliminating materials can be extended so that they last as long as the rail is in place, then they can be considered as sacrificial components of the concrete tie system and be replaced at relatively little incremental cost during routine, planned rail changeouts. However, if they must be replaced between rail replacement cycles, then a significant increase in cost will be incurred in track time, train delays, and labor effort to declip rail, raise and replace the pad, and replace the rail. Although not "solved," the rail seat abrasion problem can be managed, provided the proper combination of material is selected.

Where Do We Go From Here?

Tests at FAST require significant time and effort, thus a laboratory screening method is highly desirable. The performance of a wide range of materials (both at FAST and from a number of revenue service sites) is already known and could be used to calibrate a laboratory test machine/fixture.

In order for a laboratory test to be considered an adequate screen device, it would have to apply loads, moisture, freeze/thaw cycles and other variables that are found in the field and produce abrasion that is similar to that found in revenue service. It would also have to produce abrasion similar to that found in revenue service. These results can be used by railroads to determine which materials are most likely to last at least one rail changeout cycle before needing replacement.

In the interim, pad materials continue to be evaluated at FAST. Currently, the rail seat abrasion program is being evaluated for reconfiguration to allow improved long term evaluation of a very limited number of materials. The test conducted would be revised to allow a smaller number of material variables to be evaluated in larger sections. This will permit smaller numbers of each matrix to be measured on a periodic basis and allow long term evaluation of the most promising materials.

A more concise summary of results from the FAST abrasion test will be presented at the November 6–8, 1995 Engineering Conference to be held in Pueblo, Colorado.

Table 1. Zone 1
Concrete Tie Abrasion Experiment Materials List June '94

Zone	Tie #	Tie Type	# in Test	Hardware	MGT	Comments	Committee Suggestions
7A	870-874	FEC	5	Spring Clip/ Plastic		Installed May '94 Removed July '94	
2C	875-879	CN 60C HD-8	5	Pandrol/ Rubber		Standard as cast	
2B	880-884	CN 60C HD-8	5	Pandrol/ Urethane		Standard as cast 1 tie broken/removed	
2A(B)	885-890	CN 60C HD-8	6	Pandrol/ EVA		Standard as cast	
2A(A)	891-893	CN 60C HD-8	3	Pandrol/ Glued EVA		Standard as cast	
4B	894-899	CN 60C HD-8	6	Pandrol/ Miner pads		Standard as cast	
4A	990-904	Itisa HD-10	5	Pandrol/ DD Poly		Post tension ties In HTL since '89 Installed for abrasion 12/92	
2O	905-907	CN 60B HD-8	3	Pandrol/ Rubber		RTR #40	
2N	908-910	CN 60B HD-8	3	Pandrol/ Urethane		RTR #40	
2M	911-914	CN 60B HD-8	4	Pandrol/ EVA		RTR #40	
2L	915-919	CN 60B HD-8	5	Pandrol/ Rubber		Resurf-241	
2K	920-924	CN 60B HD-8	5	Pandrol/ Urethane		Resurf-241	
2J	925-929	CN 60B HD-8	5	Pandrol/ EVA		Resurf-241	
2I	930-935	CN 60B HD-8	6	Pandrol/ Rubber		Chemor-1984 tie	

NOTES:

2H	936-941	CN 60B HD-8	6	Pandrol/ Urethane		Chemor-1984 tie	
2G	942-946	CN 60B HD-8	5	Pandrol/ EVA		Chemor-1984 tie	
2F	947-952	CN 60B HD-8	6	Pandrol/ Rubber		Jeffamine-1984 tie	
2E	953-958	CN 60B HD-8	6	Pandrol/ Urethane		Jeffamine-1984 tie	
2D	959-964	CN 60B HD-8	6	Pandrol/ EVA		Jeffamine-1984 tie	
5C	965-974	CP CT5	10	Pandrol/ Convex rail seat/urethane		Standard as cast Special Insulators Installed 1/94	
5B	975-979	CP CT5F	5	Pandrol/ 4 Urethane 1 CRP		Standard as cast Tie 977 with CRP pad Installed 1/94	
5A	980-989	CP CT5	10	Pandrol/ Convex rail seat/urethane		Standard as cast Special Insulators Installed 1/94	
1A1	OUT OF TEST	BN 100	3	McKay/Pad cast to rail seat		Installed 1/92 - 60.5 MGT Removed 11/93	
1A(A)	990-994	CP CT3 HD-8	5	Pandrol/ Mobay		Standard as cast	
1A(B)	995-999	CP CT3 HD-8	5	Pandrol/ Dual urethane		Standard as cast	
1A(C)	1000-1004	CP CT3 HD-8	5	Pandrol/ Dual rubber		Standard as cast	
1A(D)	1005-1009	CP CT3 HD-8	5	Pandrol/ Mobay		Standard as cast	
1B(A)	1010-1014	CP CT3 HD-8	5	Pandrol/ Mobay		Jeffamine	

NOTES:

Table 1. Zone 1 (continued)

Zone	Tie #	Tie Type	# in Test	Hardware	MGT	Comments	Committee Suggestions
1B(B)	1015-1019	CP CT3 HD-8	5	Pandrol/ Dual urethane		Jeffamine	
1B(C)	1020-1021	CP CT3 HD-8	2	Pandrol/ Dual urethane		Jeffamine	
1B(D)	1022-1024	CP CT3 HD-8	3	Pandrol/ Dual rubber		Jeffamine	
1B(E)	1025-1029	CP CT3 HD-8	5	Pandrol/ Mobay		Jeffamine	
1F(A)	1030-1032	UP 497 HD-10	3	Pandrol/ Epoxy/steel plate		11/93 Pandrol repair replace 6½ mm poly pad	
1F	1033-1043	UP 497 HD-10	11	Pandrol/ 6½ mm poly		Standard as cast	
1G	1044-1049	UP 497 HD-10	6	Pandrol/ Poly		Jeffamine	
1H	1050-1056	UP 497 HD-10	7	Pandrol/ Poly		Jeffamine	
1I	1057-1063	UP 497 HD-10	7	Pandrol/ Dual rubber		Jeffamine	
1J	1064-1069	UP 497 HD-10	6	Pandrol/ Poly w/glue		Standard as cast	
1K(A)	1070-1071	UP 497 HD-10	2	Pandrol/poly w/seal ring		Standard as cast	
1K(B)	1072-1074	UP 497 HD-10	3	Pandrol/ Dual poly		Standard as cast	
1L(A)	1075-1076	UP 497 HD-10	2	Pandrol/ Poly		Installed with neoprene below poly—wore away by 100 MGT	
1L(B)	1077-1979	UP 497 HD-10	3	Pandrol/ Poly		Installed with BUNA below poly—wore away by 100 MGT	

NOTES:

1W(A)	1080-1081	K11 1011 HD-10	2	Pandrol/ 4D-2/93 4D-3/94		Standard as cast 4D pads replace double dimple EVA
1W(B)	1082-1083	K11 1011 HD-10	2	Pandrol/ 4D-2/93 4D-3/94		Standard as cast 4D pads replace double dimple poly
1X(A)	1084-1086	K11 1011 HD-10	3	Pandrol/ 4D-3/94		Standard as cast 4D pads replace rubber w/seal ring
1X(B)	1087-1088	K11 1011 HD-10	2	Pandrol/ 4D-2/93		Standard as cast 4D pads replace dual durometer rubber
1X(C)	1089	K11 1011 HD-10	1	Pandrol/ 4D-3/94		Standard as cast 4D pads replace rubber w/seal ring
1Y(A)	1090-1091	K11 1011 HD-10	2	Pandrol/ 4D-2/93 4D-3/94		Rail seat silene treated 4D pads replace double dimple EVA
1Y(B)	1092-1093	K11 1011 HD-10	2	Pandrol/ 4D-2/93 4D-3/94		Rail seat silene treated 4D pads replace double dimple poly
1Y(C)	1094-1095	K11 1011 HD-10	2	Pandrol/ 4D-3/94		Rail seat silene treated 4D pads replace rubber w/seal ring
1Y(D)	1096-1097	K11 1011 HD-10	2	Pandrol/ 4D-2/93		Rail seat silene treated 4D pads replace dual durometer rubber
1Z(A)	1098	K11 1011 HD-10	1	Pandrol/ 4D-2/93		Whole tie silene treated 4D pads replace double dimple EVA
1Z(B)	1099	K11 1011 HD-10	1	Pandrol/ 4D-2/93		Whole tie silene treated 4D pads replace double dimple poly
1O	1100-1109	BN 100	10	McKay/ Rubber		New rubber replaced Std. McKay rubber 2/94

NOTES:

Table 1. Zone 1 (continued)

Zone	Tie #	Tie Type	# in Test	Hardware	MGT	Comments	Committee Suggestions
1P	1110-1119	BN 100	10	McKay/ Sandwich		Standard as cast	
1Q	1120-1131	BN 100	12	McKay Rubber		No air entrainment New rubber replaced Std. McKay rubber 2/94	
1R	1132-1137	BN 100	6	McKay/ Rubber		Pyrament New rubber replaced Std. McKay rubber 2/94	
1S	1138-1147	BN 100	10	McKay/ 8 mm EVA		Silica fume + air	
1T(A)	1148-1149	BN 100	2	McKay/ Poly		Standard as cast Installed with Neoprene/ wore away by 100 MGT	
1T(B)	1150-1152	BN 100	3	McKay/ Poly		Standard as cast Installed with BUNA/wore away by 100 MGT	
1T(C)	1153-1157	BN 100	5	McKay/ Masti Cord		Standard as cast	
1U	1158-1167	BN 100	10	McKay/CRP glue/poly		Standard as cast	
1V	1168-1177	BN 100	10	McKay/ 8 mm poly		Silica fume + air	
2Y(A)	1178-1182	BN 100	5	McKay/ Sika flex		Standard as cast CRP rubber	
2Y(B)	1183-1186	BN 100	4	McKay/ Epoxy		Standard as cast Poly	

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Table 2. Zone 2
Concrete Tie Abrasion Experiment June '94

Zone	Tie #	Tie Type	# in Test	Hardware	MGT	Comments	Committee Suggestions
6A(C)	1673-1674	Koppers	2	Pandrol/ 2 pc sandwich poly/poly		Standard as cast Special insulator set up Installed 2/94	
6A(B)	1675-1677	Koppers	3	Pandrol/ 2 pc sandwich poly/Steel		Standard as cast Special insulator set up Installed 2/94	
6A(A)	1678-1681	Koppers	4	Pandrol/ 3 pc sandwich		Special insulator set up Installed 2/94	
3F	1682-1687	BN 100	6	McKay/ Std. EVA		Standard as cast OUT OF TEST	
3	1688-1693	BN 100	6	McKay/ Cast poly		Standard as cast Installed 2/01/93	
3D	1694-1699	BN 100	6	McKay/Steel plate cast in rail seat		Standard as cast Miner tie pads Installed 2/01/93	
3C	1700-1705	BN 100	6	McKay/ Miner pads		Standard as cast Installed 2/01/93	
3B	1706-1710	BN 100	5	McKay/ Double Crafco McKay poly		Standard as cast Installed 2/01/93 OUT OF TEST 1/94	
3A	1711-1713	BN 100	3	McKay/ Single Crafco McKay poly		Standard as cast Installed 2/01/93 OUT OF TEST 1/94	
3A(A)	1714-1715	BN 100	2	McKay/ Single Crafco Crafco rubber		Installed 2/01/93 New Crafco sealant and pad replace Crafco w/poly 1/94	

NOTES:

Table 2. Zone 2
Concrete Tie Abrasion Experiment June '94

Zone	Tie #	Tie Type	# in Test	Hardware	MGT	Comments	Committee Suggestions
2Z	1716-1725	BN 100	10	McKay/ McKay rubber		Standard as cast New rubber replace std. 2/94	
2X(B)	1726-1730	BN 100	5	McKay Epoxy & steel		More than 1.5 mm abrasion	
2X(A)	1731-1735	BN 100	5	McKay/ Sika & steel		More than 1.5 mm abrasion Epoxy repair	
2W	1736-1745	BN 100	10	McKay/ McKay rubber		New rubber replace std. 2/94	
2V	1746-1754	BN 100	9	McKay/ 3 pc. sandwich		Sandwich replaced rubber 2/94	

NOTES:

KEARNY CONNECTION—PLANNING AND IMPLEMENTATION

Part I

By: Donald O. Eisele*

Where is the Kearny Connection? What does it connect? Why is the Kearny Connection—hardly more than a dash on a large map—receiving all of the attention that it has? Why is it being built?

Why is the Kearny Connection so important that, when the Board of Directors appointed Shirley DeLibero to be the Executive Director of New Jersey Transit, the Kearny Connection was one of the specific projects that they told her they wanted to get moving? It was one of the projects that everyone talked about, but nothing ever happened. It was a project that some people told her it didn't matter what she did—she would never get it under construction. Perhaps all it needed was a woman's touch—it is not just under construction, but very near completion. Why is the Kearny Connection this important?

The answers to the first two of these questions are very simple. The answers to the other questions are far more complex.

The Kearny Connection is in New Jersey—only seven miles from Pennsylvania Station, and five miles from Hoboken. The connection is at about the point where you can see the Empire State Building out of your train window when you are riding Amtrak's Metroliner from Washington to New York City.

To put it into traditional railroad terms, this connection is at the point where the Pennsylvania Railroad's famed high line crosses over the Lackawanna's main line between Buffalo and Hoboken. It is also at exactly the same point that the westward Hudson and Manhattan track crosses under the high line on the way to the former location of the Manhattan Transfer. It is directly adjacent to the Passaic River, and it is also at exactly this same point that the New Jersey Turnpike crosses over all of this rail complex on a one hundred foot high structure.

The answer to the "why" question is much more complex. This project is the first of a series of major projects by New Jersey Transit called the "Urban Core." These projects are aimed at changing the public transportation system in New Jersey.

New Jersey Transit is the third largest transit authority in the United States. It is very different from your typical transit agency in both its scope and geography. For example, here in Chicago the C.T.A. operates the urban rapid transit lines and local buses in this immediate area. METRA operates the commuter rail lines into the suburban and exurban areas around Chicago. But neither has anything to do with Springfield or St. Louis.

New York City's mighty MTA controls all kinds of transit in the New York State portion of that metropolitan area—but has nothing to do with Albany or Buffalo. Other agencies handle matters in those areas.

New Jersey Transit has the responsibility of providing transit in every corner of the entire state of New Jersey. In fact, NJ Transit's services operate beyond the state line into the state of New York, as well as into the state of Pennsylvania. New Jersey Transit operates a fleet of two thousand buses on 124 different lines.

We also operate one light rail line—in downtown Newark, and three others are in the planning stage.

*Manager, Rail Service Planning, New Jersey Rail Operations

But that is not all. We actually own three thousand buses—not just the two thousand I mentioned a few minutes ago. New Jersey Transit supplies buses to nearly all of the private bus operators in the state. In fact, about the only scheduled buses operating in New Jersey not supplied by NJ Transit are run by Greyhound.

Of greater interest to this audience is the fact that we operate all twelve commuter rail lines in New Jersey. New Jersey Transit operates a fleet of 77 diesel locomotives, 17 electric locomotives, and 715 passengers cars. This equipment is used on 579 daily trains on a system extending over 435 route miles, and 159 stations.

Two of our lines also enter New York City, and the line from Atlantic City enters Philadelphia as well. New Jersey Transit also operates the line from Suffern, N.Y. to Port Jervis, N.Y. for the New York M.T.A.

We serve 73,000 passengers daily; 41.6 million passengers last year. We are getting close to a billion passenger miles each year on the rail system.

These lines were formerly operated by six different railroads:

Pennsylvania,

Delaware Lackawanna & Western,

Jersey Central,

New York & Long Branch

Erie, and

Pennsylvania-Reading Seashore.

We also operate over a portion of the former Lehigh Valley Railroad, just to add to the list.

Each of these roads had different operating philosophies, and vastly different marketing strategies concerning commuters. They were in competition with each other, and coordination of services to best suit existing and potential new riders was probably at the very bottom of any of their strategic plans.

By sharp contrast, in the eyes of New Jersey Transit, the services that best suit the existing and potential commuters and occasional riders is on the top of its list.

At the point in the Jersey meadows where the lines of the PRR and the Lackawanna cross, both sets of electric trains are traveling at 80 miles an hour toward New York City. But they are headed to very different parts of the city.

The business district of New York City is actually three very different areas. Each is different and attracts different industry and activities. Downtown, West Side Midtown, and the dynamically growing Upper East Side are in a very real sense three different cities. From a marketing standpoint, they are three very really different destinations for the commuting public.

The first city—the oldest area—is at the southern tip of Manhattan. This is Wall Street and the World Trade Center. This is the area to which all of the early railroad ferry services from New Jersey were directed. Terminals were built in New Jersey at Exchange Place for the PRR, Jersey City for the Jersey Central, and Hoboken for the Lackawanna. As a result of consolidations, only Hoboken is left. Seven of our commuter rail lines bring commuters to Hoboken. It is only a short ferry ride to downtown New York City.

However, there are two other, very distinct cities on Manhattan Island. The second city, on the west side of the middle of the island, is served by Pennsylvania Station. Pennsylvania Station was build in 1910 for mainline long distance trains. This second area includes Madison Square Garden, the garment center, the Empire State Building, the Javits Convention Center, Times Square and Lincoln Center.

The third city is the dynamically growing area north and east of Grand Central Terminal. It encompasses the new glass towers that line Park Avenue, Third Avenue, Second Avenue, north of G.C.T. and the United Nations. It is a whole new city growing by itself.

For passengers riding on any of the seven commuter lines in New Jersey terminating at Hoboken, getting to the West Side near Penn Station, or getting to the Upper East Side near Grand Central is a double-barrelled chore. Passengers have just had a train ride heading south and could see the city out of their train window for the past quarter hour. Now they must turn around and add twenty to thirty minutes on each trip, each day, essentially backtracking northward to their final destination.

But that is only half the rub. For the vast majority of riders, this trip back requires two additional changes of transit mode. First a ferry ride, and then a subway ride—in addition to the new original train ride to get to work. And then they must do the whole thing all over again at night to go home.

Therefore, the first and direct result of this Kearny Connection will benefit passengers from the Morristown, Gladstone, and Montclair lines of the former Lackawanna Railroad—to turn left in the middle of the Jersey meadows and reach midtown Manhattan twenty minutes sooner than they do today.

But this is only the first step.

Right behind it is another project that will be made in Montclair connecting a fourth route—allowing passengers on the Boonton line direct rail access to the Midtown area. The western terminus of the Boonton line was extended nine miles from Netcong to Hackettstown on October 31, 1994.

The Boonton line is now operated by diesel power, and as part of this project the 27,600 volt electrical system on the Morristown & Essex lines will be extended five miles to Great Notch.

On December 20, 1994, the Board approved the largest project of this series. This will be a new triple level station at Secaucus, five miles from Penn Station and four miles from Hoboken. With provision for five tracks on the upper level, two tracks on the lower level, and a giant concourse, this will allow passengers from the other three lines serving northeastern New Jersey access to the Northeast Corridor. This new transfer station allows access to the parts of New York City near Penn Station and avoids the double backing for passengers on these three lines. Figure 1 shows the location of this new station in relation to the Kearny Connection and other projects in this program.

This new transfer station will also facilitate many more transfers. Passengers from this same area will be able to use New Jersey Transit trains westward to Rahway, Princeton and Trenton on the Northeast Corridor. They will be able to connect with the southward trains to the Jersey Shore—Long Branch, Asbury Park and Bay Head.

In addition, as a result of the Kearny Connection, these passengers will also be able to transfer to trains on the three M&E lines. And finally, with the Montclair Connection, this area will be connected by rail to stations along the Boonton line.

Thus, the series of three projects—the Kearny Connection, the Montclair Connection and the Secaucus Transfer Station tie the former very separate and independent rail lines into a cohesive system. Once entering the rail system, a passenger will be able to easily go to any part of the entire system.

In Part II of this presentation, Dave Herbert will lead you through some of the challenging aspects of the Kearny construction. As complex as the engineering and construction of the actual connection is, this is only part of the project.

Earlier I mentioned that the connection is at about the point, where you can see the Empire State Building out of the train window when riding Amtrak's Metroliner from Washington to New York City. At that point you are travelling at close to 80 miles per hour, and Amtrak certainly doesn't want to have the Transit train ahead of you slow you down. Therefore, one of the design conditions was that our trains enter and leave the high line at 80 miles per hour. This leads to the requirement of twenty-six and one half turnouts. These are not exactly a stock item in your typical railroad storehouse.

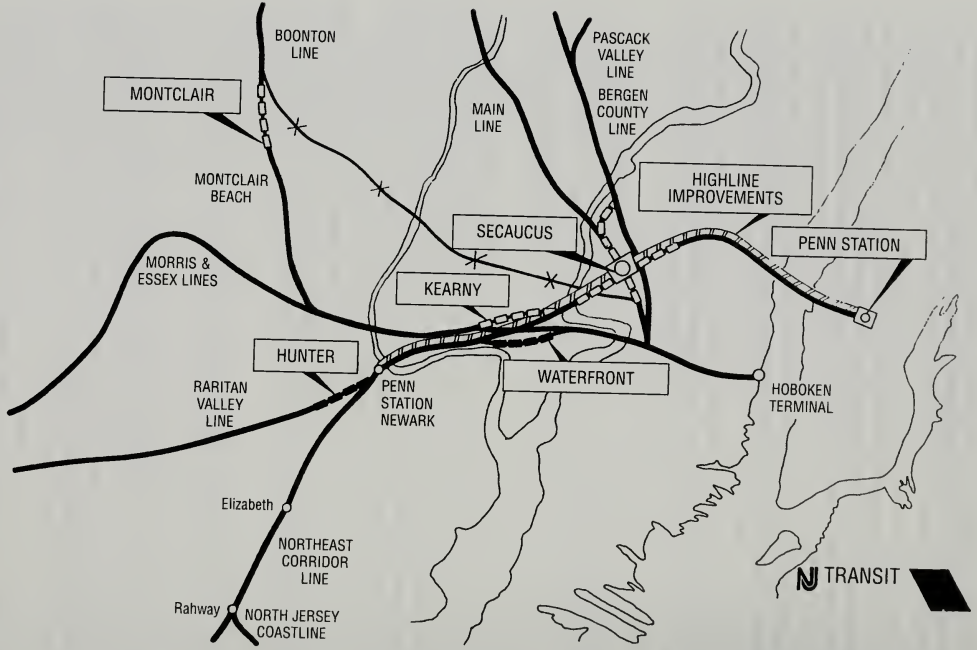
BOONTON LINE
MONTCLAIR
MONTCLAIR BEACH
MORRIS & ESSEX LINES
HUNTER
RARITAN VALLEY LINE
Elizabeth
NORTHEAST CORRIDOR LINE
Rahway
NORTH JERSEY COASTLINE
PENN STATION NEWARK
MAIN LINE
PASCACK VALLEY LINE
BERGEN COUNTY LINE
SECAUCUS
KEARNY
WATERFRONT
HIGHLINE IMPROVEMENTS
PENN STATION
HOBOKEN TERMINAL
NJ TRANSIT

Figure 1.

Electrical engineers in the audience will quickly remember that the Amtrak catenary is energized at the old 11,000 volts, 25 cycle, but the Morristown line below it utilizes 27,600 volts, 60 cycle. The transition from one electrical system to the other will be made in a fraction of a minute while travelling at high speed.

This new service to New York's Penn Station is in addition to—not instead of—the existing Hoboken based service. To operate this service, ten train sets each consisting of new ALP—44 locomotives and Comet II coaches will be required. They will be in push-pull formation, and therefore require control cab cars. These locomotives are equipped to handle the high speed voltage change.

The electric multiple-unit cars known as Arrow III's will continue to serve the Hoboken trains. Diesel powered trains will continue to operate on the Boonton line. All three types of equipment will be operating on the Morris & Essex lines.

Now that we have the connection built and equipment capable of running over it—there are a series of other parts of the project that must be considered. First, the additional equipment must be stored someplace every night. The yard at Dover is presently filled to the brim with diesel and electric trains each night.

The tiny Gladstone yard, squeezed in between a creek and the end of that single track branch, tests the ingenuity of the operating department every night to squeeze in cars for the present schedule.

How about the Montclair Branch? The station and yard at the end of the double track Montclair branch was sold to the city in 1982. This line now ends in a single stub track station with no yard facilities at all.

The answer to this dilemma is the construction of a new yard at Port Morris, nine miles west of Dover. This new eight track facility will handle the diesel hauled equipment presently stored at Dover, thereby releasing tracks for the new ALP's and Comet coaches.

Having handled the night time storage question, we must turn to the middle of the day. Some of the trains will be scheduled to reverse ends and deadhead back quickly to Dover, in order to provide the regular clock-face hourly service we desire to offer.

But the density of traffic and the operation at Penn Station is so complex, that some of the trains will have no other course of action except to spend only a few minutes allowing passengers to detrain at Penn Station, and then continue in an eastward direction under the East River. Hence these trains will be stored and serviced during the daytime at Sunnyside yard. This requires five more tracks at Amtrak's Sunnyside yard and an additional substation to handle the electrical load.

Before we pass by Pennsylvania Station so quickly, let's ask a question about the passengers. After all, moving trains is only part of the task in passenger service. More people use Penn Station in New York than the sum of all the passengers using JFK Airport, LaGuardia Airport and Newark Airport together—each day! How do these additional passengers move from these new trains to the street and their Manhattan destination?

Tracks one through four, the tracks used by many of the New Jersey Transit trains, were designed to spot Pullman cars for long distance trains and have extremely limited access. Two staircases on a long platform are not sufficient to unload trains with over one thousand passengers—all of them dashing commuters in a rush. New staircases are being built now to serve these platforms, and an entirely new access to the street level is being designed.

Parking is another part of this project, since the vast majority of suburban passengers use automobiles to reach their rail stations. Our trains are being equipped to handle wheel chairs, but a commuter cannot take his/her automobile with him/her. The typical parking lot at a commuter station is full today.

Part of this project is expanding parking facilities. If you have ever dealt with local government, you will know that this is no simple task. When a new parking lot is built—or an old one is expanded, the "Not In My Back Yard" syndrome is alive and in full force.

Finally, there is one very important long range aspect of this series of projects—the High Density Signal System.

You are well aware of the very high speed train operations in France, Germany and Japan. The trains are spaced well apart to allow this high speed. On the other hand, you are also aware of many rapid transit operations operating on two minute headways—at much lower speeds.

The railroad between Newark, N.J. and Penn Station in New York City will be operating both types of operations on the same track at the same time—both high speed and high density. When completed, this system will permit the scheduling of thirty trains per hour on each track. These trains will operate at speeds up to 80 miles per hour.

The existing signal and speed control system does not permit this density/speed combination to be operated today. A new system with nine aspects is being installed in this territory. All trains that will be operated on these tracks will be equipped to receive and respond to the nine aspects.

The Kearny Connection enters the Northeast Corridor at a new interlocking called "Swift." Swift interlocking is the first interlocking to be designed for this new high density signal system. As each new portion of the railroad between Newark and New York City is improved, it will be engineered to operate the new signal system.

For one short stretch of railroad, a great many important changes are being made to improve its capability to serve the travelling public. The Kearny Connection is the first of these.

Next, Dave Herbert will describe the challenges faced in the construction of this high speed rail connection—threading its way through a maze of columns, under traffic.

Part II

By: D. A. Herbert*

Don Eisele has given us a good introduction to the rationale and planning for the Kearny Connection Project. I will conclude by giving you an idea of the actual construction effort which has been underway for almost two years, and which will be completed this summer. I will discuss the work as it breaks down into three principal areas: Civil and Structural work, Trackwork, and Electric Traction and Signal Systems.

1. Civil and Structural

This work was the only on site construction to utilize a private contractor, and consisted of building the fill embankments and the two bridge structures to provide the roadbed for the two tracks between the Amtrak Northeast Corridor (NEC) "high line" and NJ Transit M&E line.

Track 5, the westbound connection, is 4,939 feet long, and parallels the NEC for approximately one-half its length. This posed a difficult construction operation, having to place the fill adjacent to a high speed railroad, key-in the additional fill to the existing embankment, and deal with poor access and the interference from catenary pole field guys. In addition, a large part of the fill was located on in-situ wetlands soils and had to be surcharged to avoid high initial settlements. The delay during the surcharge period had to be accommodated in the contractor's schedule.

*Construction Engineer, Bechtel Corp.

The most difficult aspect of the work was where Track 5 passes beneath the New Jersey Turnpike overhead structure, where a slab-on-grade is supported on 5' diameter caissons, socketed in bedrock some 90' deep. The first 30' had to be excavated through the existing embankment, constructed of shot rock from the Penn Station excavation in Manhattan. The contractor decided to excavate this portion by hand, in order to avoid disturbance to the adjacent NEC track, before resorting to conventional caisson sinking methods.

Track 6, the eastbound connection at slightly over 3,000 feet is shorter than Track 5. It is not located immediately adjacent to the existing NEC embankment for a long distance, as was Track 5, but this posed some distinctive problems. This embankment almost completely rested on in-situ wetlands soils, and so it also had to be surcharged. The area beneath the Turnpike on Track 6 is supported on a conventional pile bent bridge, but the alignment passes through the switchyard of the existing signal power substation, which was replaced as a part of the project (and of which I'll describe more later). This posed a major scheduling milestone to permit the completion of the Track 6 bridge. A major stream diversion and flow control effort had to be undertaken to permit construction of the bridge.

Also required as part of the Track 6 construction was modification and reinforcement of the existing bridge carrying the Amtrak "high line" over the NJ Transit M&E tracks.

2. Trackwork

Track material for the Amtrak end of the connection, designated Swift Interlocking, and for the NJ Transit end of the connection, designated Kearny Junction Interlocking was procured from two vendors. One was for the AREA design material, consisting of No. 15 and No. 20 turnouts. The other was for high speed material, utilizing No. 26.5 turnouts with moveable point frogs (MPF's) tangential geometry switches with asymmetrical section point rails, good for 80 mph on the diverging side. Some of the No. 20 crossovers are also equipped with MPF's. All material is on concrete turnout ties, and all installation is being performed by railroad forces. The Amtrak turnouts are installed over the weekend, under single track operation, utilizing the Amtrak Plasser American Switch Exchange System (SES). The NJ Transit turnouts are being built in track, as their three track line permits extended continuous outages on one of the tracks. This will be a three track universal interlocking, in conjunction with the two turnouts connecting to Track 5 and Track 6, and will be, to our knowledge, the largest all concrete tie interlocking in the United States.

3. Electric Traction and Signals

The structural steel for the new catenary structures was fabricated by a local vendor with erection accomplished partly by the civil/structural contractor and partly by railroad forces. All wire work is being done by railroad forces. The structural steel is hot-dipped galvanized to reduce future maintenance requirements. A unique feature incorporated in the catenary system of both Track 5 and Track 6 is a phase/voltage gap, where the Amtrak 11KV 25hz system interfaces with the NJ Transit 25KV 60hz system, as discussed previously by Don Eisele.

As part of the project, a new signal power system motor-generator substation was designed, built, installed and placed into service. The immediate need for this was the routing of the Track 6 alignment through the switchyard of the existing substation. These units, which have been in continuous operation since the original PRR electrification in the mid 1930's, are run by 25hz power and produce signal power at 91-2/3 hz. To maintain the future option of converting the traction power system from 25hz to 60hz, the new units utilize a solid state variable frequency drive (VFD) enabling them to operate on 60hz commercial power and put out 25hz signal power at 6.5KV. The VFD can be reprogrammed to put out 100hz signal power, if the traction power system is changed to 60hz at some future date.

To meet the demand for increased service resulting in greater train density on the Newark-New York City "high line," Amtrak and NJ Transit have embarked on a project to design and install a High

Density Interlocking Signal System (HDIS). This project will be implemented over the next several years, with the first portion to actually be designed, built, and installed for "Swift" Interlocking. The instrument houses and cases for both Swift and affected outlying locations are being procured under a design-build contract. The houses and cases are completely factory pre-wired and tested, for setting on pre-installed foundations.

The Central Instrument House for Swift, measuring 10' x 60' is to be set by the vendor, with the balance being set by Amtrak forces. As the HDIS will not be operational at the time Swift goes into service, the circuitry will be "cut-back" to interface with the existing system. All field installation and testing at Swift will be done by Amtrak forces.

The Transit signal facilities at Kearny Junction Interlocking will be designed, fabricated, installed and tested in-house by the NJ Transit signal department.

Completion of the Kearny Connection Project is scheduled for fall, 1995.

GEOSYNTHETICS IN RAILROAD APPLICATIONS

By: T. R. Baas*

AREA Committee 1 SubCommittee 10 Geosynthetics

- Geotextile Standard Specifications (Part 10)
- Geocomposite Drainage Standard Specifications (Part 10)
- Standard Specifications for Geosynthetic Products in Railroad Applications (Draft Form)
- Published for Information in May 1993 *AREA Bulletin #741* Technical Paper "Geosynthetics for Use in Retaining Walls and Slope Stabilization"
- Geogrid/Ballast Reinforcement Test Section at Fast Facility in Pueblo, CO. with AAR

Geosynthetics:

- Geotextiles
- Geocomposite Drainage Systems
- Geosynthetic Erosion Control Mats
- Geogrids
- Geocells

Geotextile Functions:

- Separation: Prevent intermixing of Ballast, Subballast, and Subgrade
- Filtration: Reduce soil piping between/within Ballast, Subballast and Subgrade
- Drainage: Providing a medium for lateral water transmission

  Benefit: Improved Drainage of Track Roadbed Section

Key Geotextile Properties:

- Strength—Installation Survivability
- Abrasion Resistance - Toughness
- Apparent Opening Size (AOS) - Soil Compatability
- Loft/Thickness - Water Transmission

Advantages Over Traditional Options:

- Ease and Speed of Installation
- Rehabilitation and New Track Applications
- Durable
- Economical
- Proven Long Term Performance

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*Manager, Allied Products, Contech Construction Products, Inc.
Presented on behalf of AREA Committee 1, Subcommittee 10

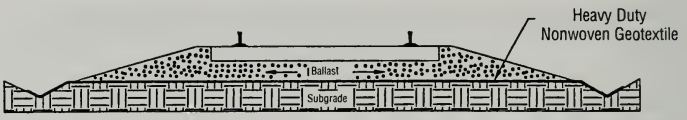


Figure 1. Rail Bed Stabilization—Nonwoven Geotextiles.

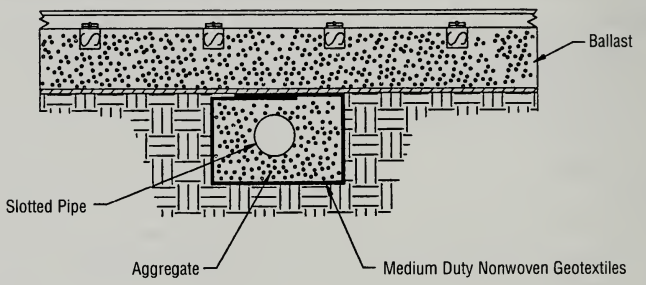


Figure 2. Subsurface Drains—Nonwoven Geotextiles.

	Track Conditions		
	Regular	**Heavy**	**Extra Heavy**
Grab Strength (ASTM D-4632)	175 lbs	225 lbs	350 lbs
Elongation at Failure (ASTM D-4632)	20%	20%	20%
Mullen Burst (ASTM D-3786)	400 psi	450 psi	620 psi
Transmissivity (at Normal Stress 3 psi) (ASTM D-4716)	2×10^{-3} ft^2/min	4×10^{-3} ft^2/min	6×10^{-3} ft^2/min
Coefficient of Norman Permeability, (k) (ASTM D-4491)	0.1 cm/sec	0.1 cm/sec	0.1 cm/sec
Permittivity (ASTM D-4491)	0.30 sec^{-1}	0.25 sec^{-1}	0.20 sec^{-1}
AOS U.S. Std. Sieve Size (ASTM D-4751)	70	70	70
Trapezoid Tear (ASTM D-4533)	100 lbs	130 lbs	150 lbs
Puncture Strength (ASTM D-4833)	110 lbs	150 lbs	185 lbs

Figure 3. AREA Geotextile Specifications for Track Applications. (All physical property requirements are *minimum* average roll values.)

Geocomposite Drain Functions:

- Groundwater Collection
- Groundwater Transportation

Benefit: Improved Drainage of Track Roadbed Section

Key Geocomposite Drainage System Properties:

- Flow Capacity—Transport Groundwater
- Compressive Strength—Resist Deformation From High Loadings
- Apparent Opening Size—Filtration/Soil Compatibility

Advantages over Traditional Underdrains

- Ease and Speed of Installation
- Rehabilitation and New Track Application
- Economical
- Proven Long Term Performance

10.4 Geocomposite Drainage System Specifications for Railroad Applications*

10.4.1 Introduction

10.4.1.1 Significance and Use

The use of geocomposite drainage systems for track roadbed stabilization applications is dependent on environmental and subgrade conditions. Geocomposite drainage systems shall be used when necessary to provide for the removal of groundwater and relief of hydrostatic pressure within the trackbed, subgrade soil and adjacent areas.

Geocomposite drainage systems have numerous civil engineering applications apart from the application described herein. Examples of these applications include turf-slope interceptor drains, retaining wall drainage, foundation drainage, roof drains and other subsurface water collection and removal applications.

10.4.1.2 Applications

This work shall consist of furnishing and installing a geocomposite drainage system as a track roadbed subsurface drain. The geocomposite drainage system shall be designed to allow for efficient collection and disposal of water while retaining the in situ soil, without clogging and/or blinding the geotextile and the core shall be designed to resist deformation, fabric intrusion and reduced flow capacity due to heavy static and dynamic railroad loads.

The use of geocomposite wick drain systems require that a complete soil investigation be made by a geotechnical engineer to determine the required flow capacity, filtration requirements and drain spacings. Geocomposite wick drain systems are not covered under this specification.

*The following pages 447–450 are excerpts from *AREA Manual for Railway Engineering*, Chapter 1, Part 10

10.4.2 Material Requirements

The prefabricated geocomposite drainage system shall consist of a flexible three dimensional synthetic drainage core. The drainage core shall be tightly encapsulated by a nonwoven geotextile. The core shall consist of a sufficient number of support members for composite support interaction between the drainage core and the geotextile overwrap to prevent geotextile intrusion and in-plane flow reduction.

10.4.2.1 Testing and Design

The long-term performance of the geocomposite drain core element shall be compatible with site-specific loadings acting upon the buried drain. The manufacturer shall provide independent test results from a qualified laboratory showing the incremental compressive creep behavior over time of the drainage core when subjected to continuously-applied pressure at equivalent to or greater than the in-service loadings determined by the design engineer.

The long-term compressive creep behavior tests may be conducted for a minimum period of 10,000 hours at 20°C. All product samples used for long-term creep deformation testing shall be indexed to their short-term compressive strength properties from quality control records. All product samples used in compressive creep testing shall be representative of the product to be supplied.

The required in-plane flow capacity of the geocomposite drainage system shall be based on the site-specific drainage requirements. Independent in-plane flow test results on representative product samples shall be provided to the designer by the geocomposite drainage system manufacturer. These test data shall include in-plane flow results in a soil environment at load levels, gradients, time intervals, and outlet spacing requirements consistent with the anticipated site-specific loadings and conditions to provide the designer with insight into the long-term in-plane flow capacity of a particular product being considered for use. ASTM D4716, Test Method For In-Plane Flow Capacity may be used as a comparison of in-plane flow capacity.

10.4.3 Geotextile Overwrap

The geotextile overwrap shall be a nonwoven, needlepunched geotextile and conform to the properties outlined in Table 10.2.1 Class A or Class B to meet the installation survivability standards of the application as determined by the engineer.

10.4.4 Packing and Identification Requirements

The geocomposite drainage system shall be provided in rolls wrapped with a protective covering. A tag or other method of identification shall be attached to each wrapped package indicating the following:

1. Manufacturer or Product Name
2. Date of Manufacture of Product
3. Roll Identification Number
4. Width of Product Rolls
5. Length of Product Rolls

10.4.5 Compliance, Inspection and Sampling Requirements

A competent laboratory must be maintained by the producer of the geocomposite drainage system, at the point of manufacture, to insure that quality control is in accordance with ASTM testing procedures. That laboratory shall maintain records of its quality control results and provide, upon request of the specifying agent prior to shipment or at any other reasonable time thereafter, a manufacturer's certificate or actual test roll data. The certification shall be based on minimum average roll values, and shall include:

(a) Name of Manufacturer

(b) Chemical Composition

(c) Product Description

(d) Statement of Compliance to Specification Requirements

(e) Laboratory Index Test Results From the Lots Corresponding To the Rolls Shipped

(f) Long-Term Compressive Creep Behavior and In-Plane Flow Test Results

(g) Signature of Legally Authorized Official Attesting to the Information Required

(h) Geotextile Overwrap Properties

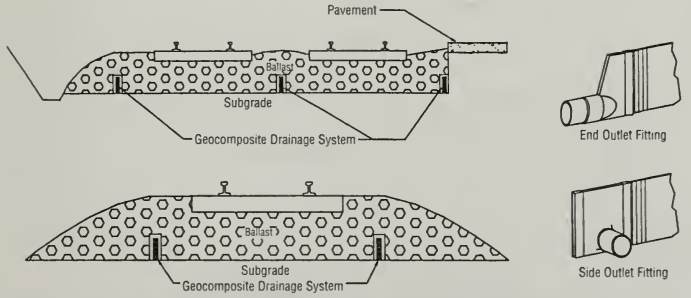
It is recommended that product samples be taken at random or at an interval specified by the Engineer in the field, and be tested at an independent competent laboratory to indicate compliance with these specifications.

10.4.6 Construction Details and Methods

The geocomposite drainage system shall be installed in accordance with the plans and specifications or as directed by the engineer.

A geocomposite drain system shall be placed in a trench as shown in Figures 10.4.6. Trench depths, widths and slopes shall be as shown on the plans or as determined by the engineer. The excavated trench material may be used for backfilling the geocomposite drain system. If required, the drain shall be temporarily supported to assure vertical alignment while the backfill is being placed. The backfill operation shall take place in a minimum of two lifts with a maximum lift thickness of 8". The final lift shall bring the backfill to an elevation approximately two inches (2") over the top of the drain. Care shall be taken during the installation procedure not to damage the geotextile or drainage core in any way.

A vibrating type compactor with a properly shaped shoe sufficient to operate within the trench limits shall be used during each backfill lift placement.



Notes
1. Depth of geocomposite drain is dependent on subsurface drainage and outlet requirements.
2. Drain depth should be adequate to avoid damage to drain from normal maintenance activities.

Figure 10.4.6 Typical Geocomposite Drainage System Locations

Fittings for the geocomposite edge drain system shall conform to the manufacturer's specifications. Fittings shall be constructed to maintain integrity of the system under rigors of installation and long term expected earth loads, train loads and traffic loads, and shall be constructed with smooth inverts to provide efficient and unrestricted outflow.

Couplings used to join rolls of drain shall meet the specifications of the manufacturer. The coupling shall provide positive connection that is sufficient to prevent joint pull apart prior to and during the backfill operation.

The spacing of outlet fittings shall be shown as on the plans or as determined by the engineer.

The diameter of the outlet pipe shall be 4" non-perforated plastic pipe with a minimum pipe stiffness of 150 psi per ASTM D2412, (Schedule 40 PVC pipe in accordance with ASTM D1785, SDR 23.5 plastic pipe in accordance with ASTM D3034 or D2751). The solvent cement for the outlet pipe and fittings shall be in accordance with ASTM D2564, D2235 or D3138. The material composition of the outlet fittings shall be compatible for direct solvent welding to the plastic outlet pipe.

All connectors between drain and fittings/couplings shall be made soil tight as recommended by the manufacturer.

10.4.7 Measurement and Payment

Geocomposite edge drain systems shall be measured by the lineal foot.

Geocomposite drainage systems shall be paid by the lineal foot of product installed and accepted by the engineer, unless otherwise prescribed by the contract.

Geogrid/Geocell Functions:

- Reinforcement of Track Roadbed Section

  Lateral restraint of Ballast and Subballast

  Improved bearing capacity of Subgrade

- Reinforcement of MSE Walls/Slopes

  Build Steeper Slopes

  Control and Use of Right-of-ways

  Benefit: Improved Performance of Track Roadbed Performance and Use

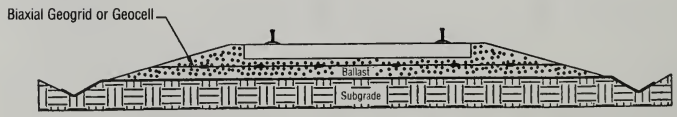


Figure 4. Rail Ballast Reinforcement—Geogrids and Geocells.

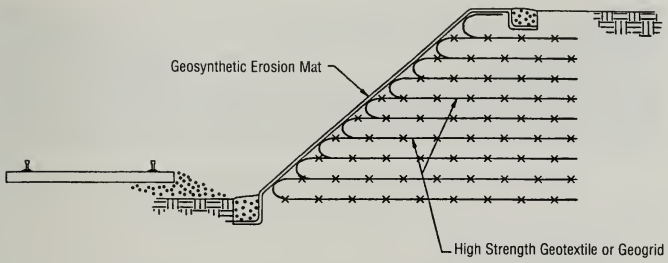


Figure 5. Steepened Slopes—Woven Geotextiles and Geogrids.

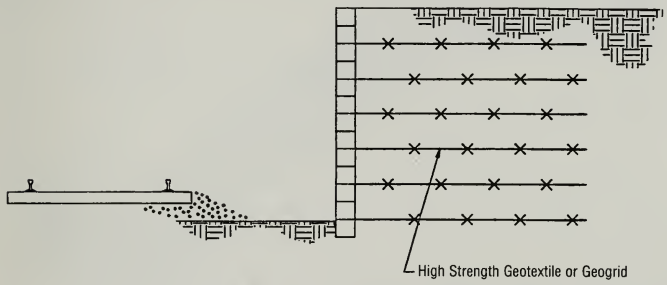


Figure 6. Modular Block Walls—Woven Geotextiles and Geogrids.

Key Properties of Geogrids/Geocells:

- High Tensile Strength—Tensile Reinforcement
- High Modulus—Spread loads over Subgrade
- Aperture Size/Stability—Provide Confinement/Reduce Lateral Spreading of Ballast and Subballast

Geosynthetic Erosion Control Product Functions:

- Seed Protection
- Turf Reinforcement
- Energy Dissipation
- Soil Containment

Benefit: Improved Land Use and Soil Loss Protection

Key Properties of Geosynthetic Erosion Control Mats:

- Tensile Strength
- Ground Cover

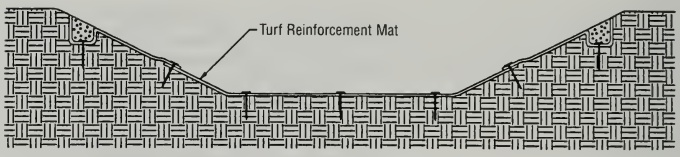


Figure 7. Flexible Channel Linings—Geosynthetic Erosion Mats.

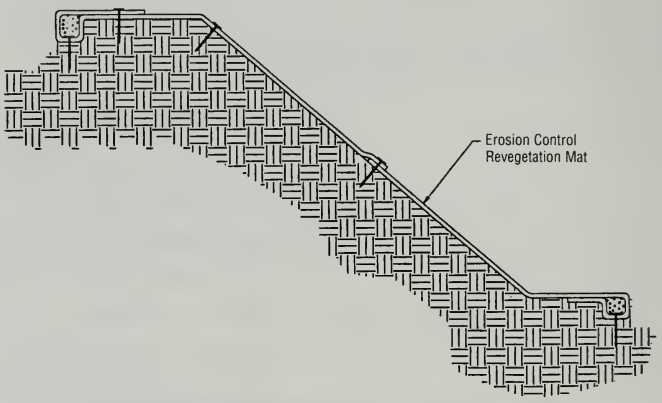


Figure 8. Steep Slope Protection—Geosynthetic Erosion Mats.

- Thickness
- Flexibility

Advantages Over Traditional Erosion Control Options:

- Long Term Durability
- Easy to Install
- Lightweight
- Predictable Performance

Product Selection and Installation:

- Design by Function/Application
- Select Product by Function
- Proper Installation - Long Term Performance

Summary:

- Geosynthetics provide effective and economical solutions and alternatives to common everyday construction and maintenance problems.
- Geosynthetic products have excellent long term performance characteristics when properly selected and installed.
- Geosynthetics can enhance the long term performance of the Track Roadbed Section.

References

Anonymous. "The Use of Geotextiles in the Prevention of Mud Pumping Upward into Railway Base," Presented at 4th Shanghai International Nonwoven Conference, Shanghai, China, Vol. 1, pp. 215–227.

Curati, M. "Effects of Railroad Track Conditions on Geotextile Performance," *Geotechnical Fabrics Report*, Vol. 3, No. 5, Sept/Oct, pp. 34–38.

Fluet, J.E. "Geotextiles and North American Railroads," *Journal of Geotextiles and Geomembranes*, 1986, Vol. 3, No. 213, pp. 201–218.

Lieberenz, K. and Chilian, G. "Geotextiles in the Layered System Under Railway Tracks," *Technische Textilien*, Vol. 29, No. 5, pp. 134–136.

Martinek, K. "Geotextiles Used by the German Federal Highway—Experiences and Specifications," *Journal of Geotextiles and Geomembranes*, 1986, Vol. 3, No. 213, pp. 17–200.

Raymond, G.P. "CN Geotextile Pack Installation," *RT&S*, May 1986, pp. 27–28.

Raymond, G.P. "Good Drainage Also Supports Crossings," *RT&S*, May 1986, pp. 52–53.

Raymond, G.P. "Geotextiles Need Good Drainage Too," *RT&S*, June 1986, pp. 22–24.

Raymond, G.P. "Performance Assessment of a Railway Turnout Geotextile," *Canadian Geotechnical Journal*, Vol. 23, No. 4, 1986, pp. 472–480.

Raymond, G.P. "Installation Factors that Affect Performance of Railroad Geotextiles," *Transportation Research Record*, 1986, pp. 64–71.

Raymond, G.P. "Research on Geotextiles for Heavy Haul Railways," *Canadian Geotechnical Conference*, Published by Canadian Geotechnical Society, Montreal, Quebec, Can. 1983, p. 3, 5, 1–3, 5, 9.

PUBLISHED AS INFORMATION
COMMITTEE 2—TRACK MEASURING SYSTEMS

Chairman: R. C. Tannahill

Assignment D1-1-93: Track Measuring Vehicle Capabilities

Features of North America's Track Measurement Vehicles May, 1995

By: Michael T. Mischke*

Information presented in this report is the result of Committee 2 Task D1-1-93 "Track Measuring Vehicle Capabilities." The purpose of this task is to produce a database of information on all types of track measuring vehicles in the United States, Canada and Mexico. The database provides a single reference source for personnel investigating capabilities of operating track measurement vehicles for purchase, contract or upgrade of existing vehicle.

The track measurement vehicle database contains over 40 entries with information fields covering,

Owner	Configuration Type	Vehicle Type	Body Type
Number of Axles	Vehicle Weight	Propulsion	Test Speeds
Crew Size	Gauge Type	Warp/Twist Type	Profile/Surface Type
Alignment Type	Curvature Type	Rail Flaw Type	Corrugation Type
Clearance Type	Rail Head Profile Type	Other Measures	Distance Measure
Event System	Track Marking	Computers	Displays
Data Storage	Printers	Video	Contact Personnel

Database format is Lotus 1-2-3 for easy distribution, display, conversion and manipulation. Plans are to update and publish the database information on a regular basis, 1–2 year cycles. The report displays the database information in a condensed form to provide good detail in a small area of print. A complete database file is available through Committee 2.

QUICK FACTS

Measurement Trends—The use of Rail Head Profile devices has dramatically increased over the past few years. Newest measurement systems focus on facility information collection (geographic location, video image, etc.) and gage restraint.

Measurement Vehicles—Class 1 railroads generally use heavy (+80 tons) rail bound coach vehicles. Transit railroads on the other hand use lighter self-propelled vehicles. All other measurement vehicles lean towards a hi-rail configuration.

Computers—A personal computer (PC, MS-DOS type) is the current choice for the data acquisition and analysis system onboard all types of measurement vehicle.

*President, Functional Data Services; Member Subcommittee 1

Features of North America's Track Measurement Vehicles

Owner/ Contractor	Vehicle Type							Geometry												Computer							
	CONF	BODY	AXLE	MOUNT	WEIGHT (in tons)	DRIVE	SPD	GAGE	XLVL	TWIST	PRF	ALIGN	CRV	Flaw	Corr	Clear	Rail	Other Measure	Dist	Event	Paint	MINI	PC	STORE	OUTPUT	Video
AGENCY																											
Association of American Railroads	H	C				S	10							U					T	M						
Association of American Railroads	R	S	2	F		S	20	W	G	C	M	M	O						T	M						
Federal Railroad Administration	R	C	4	T	75	S	80	O	G	C	I	I	R					S	T	B	1	1	2	M,F,D	A,I,V	
Federal Railroad Administration	R	C	4	T	140	T	30	W	G				R					SA	T	B	2		1		V	
U. S. DOT	R	C	2	F	26	S	50	W	G	M	M,C	M,C	M						T	B		1		M	A,I	
U. S. DOT	H	O	2	F	5	S	10							U					T	M			1	F	V	
CONTRACTOR																											
ENSCO	R	S	4	T	140	T	80	O	G	C	I	I	R						T	B			2	D	A,I,V	
E. H. Reeves	H	O	2		5	S	30	R									Y		T	M			2	F,R	L	
Pandrol Jackson, Inc.	H	O	2		14	S	25								C				O	M		1			I,V	
Sperry Rail Services	R	S	4	T	80	S	13							U,I							5	1	1	F	A,O	
Sperry Rail Services	H	O	2		13	S	13							U							5	1	1	F	A,O	
Trak-Tech	H	O	2	T	14	S	25	W	G	C	M	M	M					R	T	B		1	2	M,R,D	I,V	
RAILROAD																											
Amtrak Railroad	R	S	4	T	64	T	150	P	G	C	I	I	R					C,R	T	B		1	2	M,F,D	A,V	Y
Amtrak Railroad	R	C	4	T	50	S	50											C,O	O	B			1		I,V	
Burlington Northern Railroad	R	S	6	T	105	T	60	S,R	G	M	M	O	R				Y	D,R	T	M	1	2	2	M,D	I,V	
C N North America	R	C,O	4	T	17	T	90	L	O	C	I	I	O		I		Y	G,H,R,O	T	B		1	2	M,F,R,D	A,I,V,E	Y
Canadian National Railway	R	C	2	T	15	S	19							U							1		2	R	I,V	
Consolidated Rail Corporation	R	S	4	T	76	T	90								I		Y	H	T	M		1	3	M,R,D	A,I,V,L	
Consolidated Rail Corporation	H	C	2	T	4	S	25									L		O		M			4	F		
Consolidated Rail Corporation	R	S	4	T	133	T	90	R	G	C	I	I	R						T	M	1		5	M	L,V,E	Y
CP Rail System	H	C	2	T	12	S	15	O						U					T	B	2		1		V	Y

Features of North America's Track Measurement Vehicles

Owner/ Contractor	Vehicle Type							Geometry												Computer						
	CONF	BODY	AXLE	MOUNT	WEIGHT (in tons)	DRIVE	SPD	GAGE	XLV	TWIST	PROF	ALIGN	CRV	Flaw	Corr	Clear	Rail	Other Measure	Dist	Event	Paint	MIN	PC	STORE	OUTPUT	Video
CP Rail System	R	S	4	T	109	T	60	0	G	C	M,I	O	R		L			D	T	B		1	7	M,F,R,D	A,I,V	Y
CP Rail System	H	C	2	T	12	S	15	L						U					O	M	2			D		
CP Rail System	R	S	6	T	117	T	60	L	G		M,I	O	R				Y	G,D	T	B		1	3	M,F,R	A, I, V	Y
CSX Transportation	H	C	2		12	S	15							U					T	M	1		2	R	V	
CSX Transportation	R	S	6	T	93	T	60	M	G	M	M		R				Y	D,H,R	T	B	1	1	5	M,F,R,D	I,V	Y
CSX Transportation	H	O	2		5		40	W	G	C									T	M	2				A,I	
Long Island Railroad	R	C	4	T	58	S	60	S	G	C	I	M	M					T,R	T	B	1	1		M,D	A,I,V	Y
Norfolk Southern Railroad	H	O	2		10	S	3									F			T	M			1	R	E,V	
Norfolk Southern Railroad	R	S	6	T	144	T	70	R	G	O	O		M				Y		T	M	1	1	3	M	I,V	Y
Norfolk Southern Railroad	H	O	2		13	S	12							U					T	M	2		1		V	
Santa Fe Railway	R	S	6	T	55	T	70	R	G	C	M	M	M				Y	D,R	T	B	2		4	F	E,V	Y
Southern Pacific	R	S	4	T	140	T	80	O	G	C	I	I	R						T	B			2	D	A,I,V	
Union Pacific Railroad	H	C	2	F	7		25	W	G							L			T				3		O	
Union Pacific Railroad	R	C	2	F	40	S	50	W	G	C	M	M	M					H	T	B	1	1	1	M,D	I	Y
Union Pacific Railroad	H	O	2	T	13	S	15													M	2		2	F	V	
Union Pacific Railroad	H	O	2	F	4	S	40						M					G	O	B	1		1	D	E,O	Y
Union Pacific Railroad	H	C	2	F	5		2	O	O							L		S	T				1	F	O	
TRANSIT																										
Mass. Bay Trans. Authority	H	C	2	T	14	S		W	G	C	M	M	M							B	1		1	F,R	A,V	
N. Y. City Transit Authority	R	C	6	T	50	S	70	S	G	C	M	M	M						O	M		1			A,I,V	Y
N. Y. City Transit Authority	R	C	6	T	51	S	70	S	G				M		I	L		T,O	O	M		1			A,I,V	Y
Port Authority Trans Hudson(PATH)	R		4	T	50	T	55	R	G	C	I	I	O						T	B			1	R,D	A,V,L	
Southeastern PA Trans. Authority	R	C	2	F	32	S	50	W	G	C	O	O	O					R	O	B			1	F,R	I,V	
Southeastern PA Trans. Authority	R	C	4	T	50	S	30											C,O	T	B			2	R	I,V	

Key to Features of North America's Track Measurement Vehicles

Vehicle Type	Geometry					
CONF	**GAGE**	**FLAW**	**CORR**	**CLEAR**	**RAIL**	
R = Rail	W = Wheel	(Rail Flaw)	(Corrugation)	(Clearance)	(Rail Profile)	
H = Hi-Rail	S = Sword	U = Ultrasonic	I = Inertial	F = Feeler	Y = Yes	O = Other
	L = Optical	I = Induction	L = Optical	L = Laser		
BODY	M = Magnetic	M = Magnetic	C = Contact			
S = Coach	R = Rail Profile	**Other Measure**				
C = Custom		G = Grade		**DIST**	**EVENT**	**PAINT**
	XLV (Crosslevel)	D = Dip/Sag/Hump		(Distance)		
AXLE	G = Gyro	T = Third Rail		T = Tachometer	M = Manual	Number of
# of Axles	**TWT** (Twist/Warp)	C = Catenary Wire			A = Automatic	
	M = Mechanical	H = Hot Box			B = Both	
MOUNT	C = Calculated	R = Ride Quality				
T = Truck	**PRF** (Profile)	SA = Split Axle				
F = Fixed	M = Mechanical	S = Survey/GPS				
	I = Inertial	**Computer**				
WEIGHT	C = Calculated	**MINI**	**PC**	**STORE**	**OUTPUT**	**VIDEO**
(In Tons)	**AL** (Alignment)			(Storage)		
	M = Mechanical	Number of	Number of	M = Tape	A = Analog Chart	Y = Yes
DRIVE	I = Inertial			F = Fixed Disk	I = Impact Printer	
T = Tow	C = Calculated			R = Removable	E = Electrostatic	
S = Self	**CRV** (Curvature)			Disk	L = Laser	
	M = Mechanical			D = Digital	V = Display	
SPD = Max. Speed	R = Rate Gyro					

A DISCUSSION OF THE PAPER ENTITLED "FATIGUE RELIABILITY ANALYSIS PROGRAM FOR RAILROAD STEEL RIVETED BRIDGES"[1]

By: Dr. Robert A.P. Sweeney* and Dr. John W. Fisher**

The authors have gone through an interesting exercise in attempting to layout a methodology for reliability analysis of steel riveted bridges. Given the limited loading data available, an SD-60 type train and a E-55 type train, the exercise is of value to those interested in the methodology of this type of problem. The specific conclusions are not likely to be correct. By this, I mean that the real reliabilities are probably much different than those calculated. A number of points of discussion follow:

On page 194, Equivalent Number of Cycles: AREA, in Article 7.3.4.2(g) (1994), recommends a different formula for evaluating effective stress range, where a route-mean-cube effective stress range is calculated to be used with the total known number of cycles. The authors in formula 9 have used the reverse in using the highest or primary stress range and then calculating an equivalent number of cycles. Do both lead to the same result? Under what circumstances are the two methods not analogous? The AREA method assumes a rain flow counting method for cycles.

The Schilling procedure[2] (formula 9) takes the cycles from a single highway truck and decomposes them into a primary cycle with superimposed secondary cycles from vibration. The question needs to be raised as to its relationship to a static analysis of the train with design impact. It appears as though the authors consider only the primary cycles from the static analysis. This approach does not consider the need to calibrate the analysis with field measurements.

The value of "b" used in formula 9 does not seem to be in the paper. Is it different for each data set referred to in item 1 on page 192, or have the authors used the same value throughout? The parameter "c" mentioned on page 206, item 6, is also part of this. Did the authors use the AREA values of $b = -3.0$ and $c = 2.183 \times 10^9$ for Category "D" and $c = 4.446 \times 10^9$ for Category "C"?

On page 198, Fatigue Reliability Model Routine: In applying the 7 ksi (48 MPa) limit, have the authors considered the 0.1% criteria mentioned in AREA Article 7.3.4.2(g) (1994)? In other words, do the authors recognize damage caused by lesser stress ranges when a significant number of higher stress range cycles get the crack moving? This seems particularly important in view of the use of the highest or peak stress range through formula 9.

Unfortunately, while the authors were writing this paper, a major test program on full size beams, girders and hangers conducted at Lehigh University[3] and at the University of Alberta[4] sponsored by CN[5] has altered the fatigue curves for riveted structures. The results of the testing are to be published (Zhou 1995) in the Proceedings of the 12th Annual International Bridge Conference sponsored by the Engineers' Society of Western Pennsylvania held in June 1995.[6] Of particular importance is the indication of a variable amplitude fatigue limit of 6 ksi and not 7 ksi constant amplitude using root-mean-cube equivalent stress ranges.

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*Assistant Chief Engineer—Structures, Canadian National (CN) and Vice Chairman of Committee 15

**Director, ATLSS Engineering Research Center, Professor of Civil Engineering, Lehigh University, Member Committee 15

[1] E. Manos Maragakis and David B. Sanders, American Railway Engineering Association, Proceedings Volume 96, *Bulletin 751*, May 1995, pp. 191–209.

[2] Reference 22 in author's paper.

[3] Y. Edward Zhou under the direction of J.W. Fisher, B.T. Yen and R.A.P. Sweeney.

[4] Danny Adamson and Jefferey DiBattista under the direction of G. Kulak.

[5] Canadian National Railways.

[6] Zhou, Y. Edward., Yen, Ben T., Fisher, John W., Sweeney, R.A.P.. 1995. "Examination of Fatigue Strength (Sr-N) Curves for Riveted Bridge Members." Proceedings of the 12th Annual International Bridge Conference sponsored by the Engineers' Society of Western Pennsylvania.

On page 202, the authors state that "the calculated reliability is based on the initiation of a crack and not failure of a member or a connection." Given the data quoted on page 192, item 1, which was the basis of the authors' calculations, this does not seem to be correct. If the authors used all the data, there is a mix of visual crack detection (not crack initiation) and section failure data.

AREA in its specifications had only used the Section failure data, which remains Category "D" for stress ranges exceeding 9 ksi[7] (Adamson 1994).

The term initiation of a crack in fracture mechanics terms is rarely relevant to civil engineering structures, as the steel used and fabricated in these applications generally has initial flaws, so the process is one of crack propagation.

Given the possible errors mentioned above, the authors then calculate the reliability after the year 2028 as 98.22%. It is the writer's understanding that a reasonable safe bridge structure should be able to at least be two standard deviations from the mean of failure, which means a probability of failure of 2.5% or a reliability of 97.5%. The authors' calculation is close to that limit for the girder.

Clearly the truss hanger reliability of 72.83% is unacceptable.

The appropriate conclusion might be, given the imposed assumptions, that the girder is adequate for the time period assessed, but that the truss hanger is not. The implication, on page 204, that extensive and frequent inspection is required, is not enough. The member should be retrofitted, strengthened or replaced long before its reliability deteriorates to such a low level.

It would appear that figures 9 and 10 are reversed. Figure 9 shows the 90 MPa (13 ksi) stress range and Figure 10 shows the 60 MPa stress range. The text, on page 203, says that the 90 MPa stress range was under the SD-60 Diesel train which is labeled as Figure 10.

[7]Proposed change for next issue of Chapter 15.

DYNAMIC PERFORMANCE OF RAIL SEAT PADS SUBJECTED TO IMPACT LOADING

By: Nianzhi Wang*, William J. Venuti** and Sidney Mindess*

Introduction

Impact loads applied to concrete ties, due primarily either to the wheel flats of a train or to rail abnormalities, may cause the ties to crack even after only a few months in service[1,2]. These impact loads can be up to 3 times higher than the normal quasi-static loads on the ties. An impact factor, recommended by American Railway Engineering Association for use in the design of concrete ties, has been increased from 50% in the 1970's to the current 200%[3]. One of the major measures taken to attenuate the effect of these impact loads on concrete ties has been the use of rail seat pads with an increased resiliency. The rail seat pad, designed to prevent abrasive wear of the concrete due to small longitudinal and transverse movements of the rail, is the component of the rail fastening system placed between the rail and the tie. It also provides electrical insulation for signal circuits and serves as an elastic medium to attenuate impact forces. However, there is currently no standard method available which can be used to evaluate the dynamic characteristics of the pads. Meanwhile, there is evidence[1] that the more resilient pads may either deteriorate with repeated loading or undergo hardening under impact loads in service, leading to even more severe damage to the ties. This has long been a major concern, but studies of this problem have been very limited.

Since 1980, when Dean, et al.,[1] at the Battelle Columbus Laboratory developed a basic apparatus to evaluate the 'resilience' of the pads under impact, some research has been carried out based on this method. Grassie[4] compared data from laboratory tests on rail pads using a Battelle style apparatus and data from field experiments. He found that although the performance of the pads varied with the particular track conditions, such as ballast support conditions and the magnitude of the quasi-static loads applied, the ranking of the dynamic performance of the rail pads was identical in both the laboratory and field experiments, and there was an excellent correlation between the laboratory and the field data. Igwemezie and Mirza[5] also used a similar method to test pad performance under impact. They found that the impact load versus peak strain of the concrete tie relationship was linear. The impact load and hence the tie strain was a logarithmic function of K*, which was a product of axial stiffness and the shape factor of the pad. However, Dean, et al.,[1] in 1983 tested about 20 types of pads and found that the measured static pad stiffness provided a very unreliable measure of strain attenuation capability. Their rankings of the static stiffness and the strain attenuation capability were quite different. Since then, many new types of pads have been developed, but there has been no recent work on systematically testing a series of different pads that are currently being used. Also, more study is particularly necessary on the deterioration of the dynamic performance of pads in service.

Research Objectives

Thirteen types of pads, including both new and used pads, had been collected and tested for their static compressive stiffness at San Jose State University by Venuti[6,7] in 1992. In the present study, the impact characteristics of these pads were determined, using a fully instrumented impact machine to produce impact loading. For each pad, the attenuation of the impact load, the concrete rail seat strain at the impacted end of the tie and the concrete rail seat strain at the nonloaded end of the tie were measured and ranked. A frequencies-domain analysis was carried out to determine the underlying relationship amongst these three parameters. In addition, the reduction of the attenuation capability of two types of commonly used pads, after 3.5 years in service, was evaluated and analyzed.

*Professor, Department of Civil Engineering, University of British Columbia, Vancouver, Canada
**Professor, Department of Civil Engineering, San Jose State University, San Jose, California

Experimental Procedures

1. Impact Machine

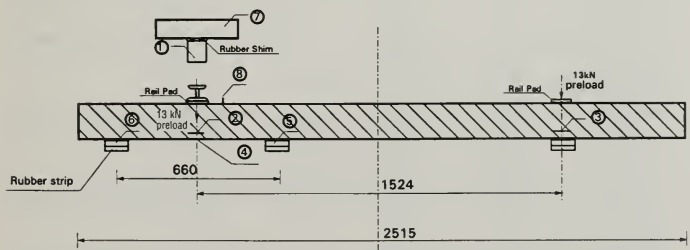
In order to simulate the response of ties to impact loading, it was considered essential to duplicate, as closely as possible, the impact pulse measured in a track in service, in terms of both the shape of the load vs. time curve and the duration of the impact event. An instrumented drop-weight impact machine, designed and built at the University of British Columbia, was used. This machine could produce an impact pulse with 3–5 ms duration, which is considered to be the typical duration of the impact event due to wheel flats, and is the type of pulse which causes the most serious damage to the concrete tie. With this machine, the hammer weight, and the type and thickness of the rubber shim between the hammer and the striking tup are adjustable (see Figure 1), to permit the production of a specified duration of pulse on the tie over a certain range. The drop height of the hammer will influence the amplitude but not the duration of the pulse. In this study, a 60 kg hammer weight was chosen and no rubber shim was used. With this arrangement, the duration of the first pulse of concrete strain was less than 2 ms, and the duration of the impact load was about 4 ms, coinciding with the pulse measured in track[1].

Spectral analyses showed that the first three bending modes of the tie in these tests were 98, 358 and 618 Hz, quite close to the measured bending modes in the field: 131, 356 and 638 Hz. The very low magnitude of the first mode in the laboratory can be attributed to the single-sided loading, in contrast to the two-wheeled loading of the tie in track under service load conditions[1].

2. Instrumentation and Setup

Figure 1 shows the experimental setup designed for a series of tests for an other project related to the improvement of the dynamic properties of the concrete ties. For the tests on the pads described here, only channels 1, 2, 3, 4 and 8 were utilized.

A standard 136 RE rail segment, 292 mm (11.5") in length and 21.6 kg (47.5 lb) in mass, was fastened to one rail seat of the tie using a Pandrol fastening system with the pad to be checked between the rail and the tie. The hammer was released from a predetermined height and dropped onto the top of the rail segment, inducing an impact pulse. A new Acme 84-006 EVA pad was placed on the other rail seat only as a shim for the application of the preload. When the testing for each pad at the impacted end was finished, the EVA pad at the free end was also replaced with a new one. Two strain gauges*,100 mm (4") in length, were affixed to the vertical surfaces of the concrete at the two rail seats, 20 mm from the bottom of the tie. A crack gauge mounted on the bottom of the tie was used



Ch. 1: load cell on tup; Ch. 2: strain gauge; Ch. 3: strain gauge; Ch. 4: crack gauge; Ch. 5: load cell on support; Ch. 6: load cell on support; Ch. 7: accelerometer on hammer; Ch. 8: accelerometer on tie. Stress coat applied on the opposite side of the tie at rail seats.

Figure 1. Layout of Impact Test on Concrete Railroad Ties.

*Model EA-66-40CYB-120, manufactured by Measurements Group, Inc. USA.

to determine whether the concrete tie cracked at all during the impact event. A small preload, about 13.4 kN (3 kips), was applied to both rail seats to prevent the tie from rebounding, just as the rail under a traveling train would restrain a single tie after an impact pulse. The striking tup at the bottom of the falling mass was instrumented with electric resistance strain gauges, and thus acted effectively as a load cell. A PC-based high speed data acquisition system was used to collect the data from all channels, at time intervals of 30 μs. A more detailed description of the load cell and the data acquisition system is given in Reference 9. The support span for the centered impact load on the rail seat was set to 660 mm (26"), which is consistent with the setup for static tests of concrete ties recommended by the AREA Manual[3]. The rubber strips used on the tops of the anvils were of neoprene rubber, 406 mm (16") long, 51 mm (2") wide and 19 mm (3/4") thick. The durometer-A reading for the strips was 60.

3. Specimens

a. *Prestressed concrete tie*

The concrete tie used was a Dywidag UP-2 tie, provided by ITISA of Mexico. Such ties are used for railroad systems in the USA and conform to the AREA Manual[3]. The concrete strength at 28 days was 65 MPa (9.42 ksi). The initial prestress force was 40,000 kilograms, with 14% loss assumed. The design bending moments at the rail seat were 30.18 kN.m (267 inch-kip) positive and 16.1 kN.m (142 inch-kip) negative. The ultimate positive bending moment was 45.33 kN.m (401 inch-kip). The cracking strain at the bottom of the rail seat, under static bending moment, was 369 microstrain by calculation, and actually measured 429 microstrain during the static calibration.

b. *Rail seat pads*

On the basis of the work done by Venuti[6,7], the pads tested are described below:

Rail Seat Pads Tested for Impact Attenuation

No.

1. (JM 1) 5.0 mm Amtrak Acme 81-020 EVA 1240 18% vinyl, flat pad.
2. (JM 2) 6.5 mm Amtrak Acme 84-006 EVA 18% vinyl, double dimple.
3. (JM 5) 6.5 mm CSX Acme 90-002/P Texin 445 Polyurethane, double dimple with sealing rings.
4. (JM 7) 5.0 mm CN CRP-Pandrol 6252 reinforced dual durometer rubber, double dimple with one sealing ring.
5. (JM 9) 7.5 mm CSX CRP reinforced dual durometer rubber, double dimple with sealing rings. In service on Burlington Northern Railroad.
6. (EVA 4) 6.5 mm Acme (Pandrol) EVA double dimple.
7. (EVA 1) 6.5 mm Acme (Pandrol) EVA double dimple. After 185 MGT (million gross tons tonnage) in service on Southern Pacific track at Dragoon.
8. (LUP 4) 6.0 mm Lupolin flat pad, EVA, manufactured in Germany.
9. (LUP 1) 6.0 mm Lupolin flat pad, EVA, manufactured in Germany. After 165 MGT in service on Southern Pacific track at Dragoon.
10. Safelok 8.0 mm rubber with 3 chevrons on top surface and 4 chevrons on bottom surface, has 4 locating tabs. Similar to the type installed in Southern Pacific track at Dragoon in January, 1991, and in Burlington Northern Railroad track at Hemingsford.
11. (CRP 1) 7.0 mm CRP rubber reinforced dual durometer, double dimple. Sealing rings on each side.

12. 6.0 mm CRP flat pad with rubber bottom and polyurethane top. Similar to pads used on Canadian National Railway. (Has small 5 mm hole in center.)
13. 8.0 mm CRP polyurethane pad with 1 mm rubber bottom. Polyurethane top has 10 grooves. Rubber bottom is flat. Has 4 locating tabs.

The static stiffnesses and durometer numbers of the pads are listed in Table 1, where E1 is the pad stiffness determined from the slope of the load vs. deflection curve connecting the points at loads from 17.8 kN (4 kips) to 89 kN (20 kips) and E2 is determined from the slope of the load vs. deflection curve from 106.8 kN (24 kips) to 195.8 kN (44 kips). E1, rather than E2 was used as a reference in this test because the starting point of E1, 17.8 kN, is almost the clamping force on the pads due to the Pandrol fastening system and 89 kN is very close to the maximum impact load reached during this test series.

4. Testing and data processing

The drop heights of the hammer were chosen as 102 mm (4"), 203 mm (8") and 305 mm (12"). Every pad underwent three blows at these three drop heights. The impact load histories, the concrete strains at the two rail seats, and the accelerations of the tie were measured and recorded for each impact blow. The data were later digitally filtered using a PC signal processing software, MATLAB. The cutoff frequency used was 2000 Hz, which was found to be quite good for these tests; most of the energy in the first four bending modes was preserved. Pad no. 1 was used as a datum to evaluate the characteristics of all of the other pads.

For each pad tested, the maximum values of the data obtained from each of the three drop heights were compared to the maximum values for pad no. 1 obtained from the same drop height. For example, the strain attenuation for pad no. 2 was estimated by:

$$A_2 (1 - \epsilon_2/\epsilon_1) \times 100\%$$

Table 1. Static Stiffness of Rail Seat Pads
E1: 17.8 to 89 kN (4 to 29 kips)
E2: 108.8 to 195.8 kN (24 to 44 kips)

Rail Pads No.	E1 kN/mm	E1 kips/in	E2 kN/mm	E2 kips/in	Durometer -A
1	1120.0	6,400	1312.5	7,500	96
2	210.0	1,200	630.0	3,600	94
3	227.5	1,300	262.5	1,500	89
4	227.5	1,300	577.5	3,300	79 & 73 (*)
5	122.5	700	542.5	3,100	77 & 65 (*)
6	210.0	1,200	490.0	2,800	98
7	315.0	1,800	1400.0	8,000	95
8	560.0	3,200	1400.0	8,000	100
9	490.0	2,800	1242.5	7,100	96
10	63.0	360	210.0	1,200	84
11	140.0	800	420.0	2,400	82 & 72 (*)
12	187.3	1,070	521.5	2,980	97 & 65 (*)
13	148.8	850	306.3	1,750	96 & 63 (*)

*First number is the durometer at top surface, second number is for bottom surface.

where, ϵ_2 and ϵ_1 are the maximum strains for pad no. 2 and pad no. 1 respectively, due to the impacts under a particular drop height of the hammer. The final value of strain attenuation listed for pad no. 2 is the average of A_2 values for the blows under the three drop heights of 102 mm, 203 mm and 305 mm.

Results and Discussion

1. Rail seat strain—impacted end

Figure 2 shows the rail seat tensile bending strain attenuation for the impacted end. The static stiffnesses of the pads, EI, are shown together as a reference. The typical strain history diagrams for the impacted and nonloaded ends are shown in Figure 3. In general, the strain attenuation capabilities of the pads correspond roughly to the pad static stiffnesses, but there are some exceptions, such as with pads nos. 5, 12 and 13. In other word, the pad static stiffness could be used as a reference but not as a criterion to evaluate the properties of pads under impact.

The frequency domain analysis of pad properties shows more clearly the role of a resilient pad in attenuating the impact load. From Figure 4 it may be seen that pad no. 10 acted as a low- pass filter, which filtered almost all of the energy in the third and fourth tie bending modes and nearly half the energy in the second mode. The low-pass "break frequency," the most important filter characteristic, seems to be near the second bending mode resonance frequency for pad no. 10. Accordingly, the time duration of the strain in one cycle for pad no. 10 was apparently extended, as shown in the strain history diagram in the time domain in Figure 3.

Since all of the energy consumed in the tie during the impact event was much less with pad no. 10 than with pad no. 1, special attention should be paid to the deterioration or hardening of pad no. 10 which may absorb more potential energy from the falling mass. Actually, the much larger hysteresis loop in the load-deflection curve for pad no. 10 compared to the loop for pad no. 1 in the static tests, as shown in Figure 5, has already given such a warning.

It should be noted that although the evidence shows that the results of impact attenuation tests of pads in the laboratory were consistent with the results obtained by measurements made under moving loads in track[1,3], there is no universal correlation between laboratory and field tests that can be applied to every type of pad.

2. Rail seat strain—nonloaded end

Figure 6 shows the *compressive* rail seat strain attenuation at the nonloaded end for all pads under impact. As was seen in Figure 3, the maximum tensile strain occurred at the rail seat at the impacted end, while the maximum compressive strain occurred at the bottom of the tie under the rail seat at the nonloaded end, implying that quite a high tensile strain may be reached at the top of the tie, where the flexural strength for negative bending moment is only about half of that for positive bending moment. The strain attenuation effect of the pads on the nonloaded end is thus the other important contribution of pads to the protection of concrete ties, particularly when the impact load is excited due to rail defects that occur at only one side of the track. It should be noted that under service load conditions in track, both rail seats are subjected to impact from wheel flats simultaneously, since wheel flats are somewhat identical on both wheels of a given axle. However, field tests have shown that owing to excitations, a rail seat can be subjected to both positive and negative moments under impact loading. (See Figure 3, as an example).

The ranking of the strain attenuation capabilities for the nonloaded end, shown in Figure 6, was similar to the ranking for the impacted end, shown in Figure 2. However, a remarkable change was seen for pad no. 9, which attenuated the tensile strain at the impacted end by 11% but increased the compressive strain at the nonloaded end by 2%. As shown above, a resilient pad such as pad no. 10 usually acted as a low-pass filter, which filtered the energy in all of the bending modes which are higher than a particular low-pass "break frequency." However, the dynamic behavior of pad no. 9 is

quite different (see Figure 7). It can only filter a selective vibration mode, instead of all of the higher bending modes. Although the energy in the third bending mode was significantly filtered, the energy in the fourth mode actually became a little higher.

The mode vibrating shapes of concrete ties had been investigated previously[4,10]. For the similar types of the concrete ties, the mode shapes are generally the same, with a small variation in the resonance frequency for each mode. They are illustrated in Figure 8 with the corresponding frequencies measured in the present tests. Since the initial condition of all mode shapes was determined by the impact pulse which forced the rail seat at the impacted end downward, the mode shapes at the beginning of the impact pulse era should be the same as the shapes illustrated in the figure, and both the tensile strain and the compressive strain reached a maximum within the first cycle. When the tie vibrated in its first mode, the dynamic tensile strain in the tie was a maximum at its center, having little influence on the rail seat strain. The second and forth modes are asymmetrical. When the tie vibrated in these modes, the tensile strain response of the tie was a maximum near the bottom of the rail seat at the impacted end, but the compressive strain response of the tie was a maximum near the bottom of the rail seat at the nonloaded end. With pad no. 9, the second and fourth modes of the tie were not attenuated, but were increased somewhat, contributing to a small increase of the compressive strain at the bottom of the nonloaded end. The third mode (618 Hz) played an important role in the tensile strain response of the tie at both rail seats since in this vibrating mode the maximum tensile strain occurred right at the rail seats. The significant attenuation of this mode by pad no. 9 reduced the tensile strain of the tie at the bottom of two rail seats, contributing to the reduction of total apparent rail seat tensile strain at the impacted end. However, since the apparent strain response is the sum of the strain response contributions of each mode, the change in the third mode may reduce the tensile strain fraction in the total response at the bottom of the nonloaded end, leading to a further increase of apparent compressive strain.

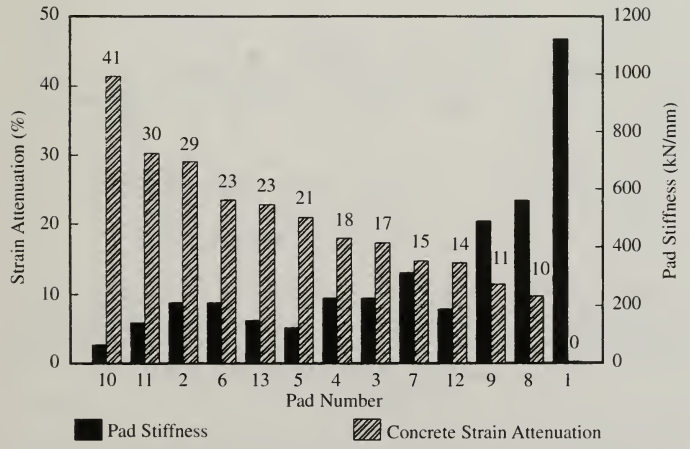


Figure 2. Pad performance under impact loading—Rail seat strain attenuation (Impacted End).

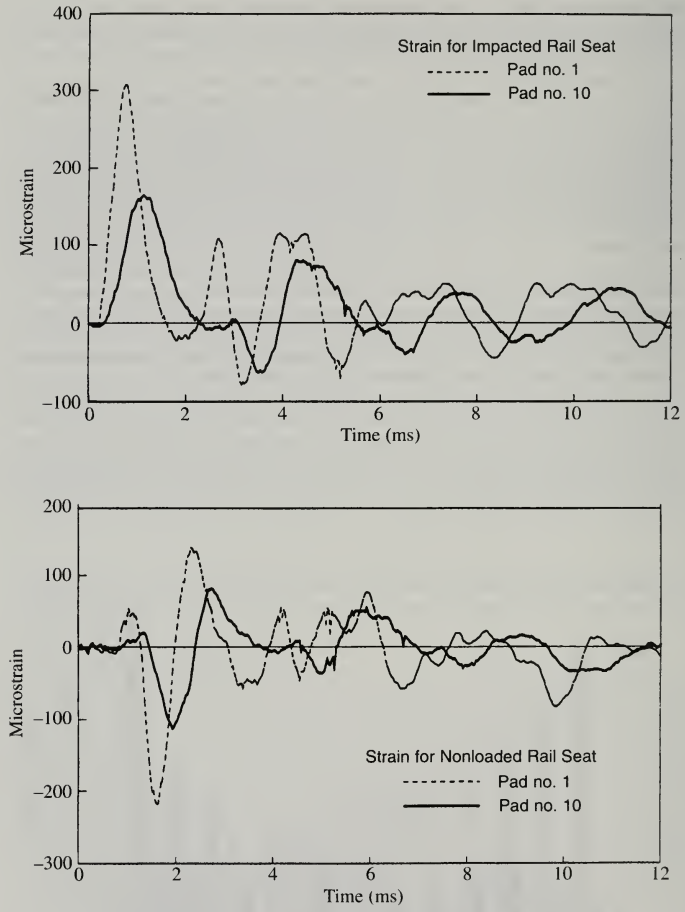


Figure 3. Rail seat strain history for Pad no. 1 and no. 10—Drop Height = 305 mm (12").

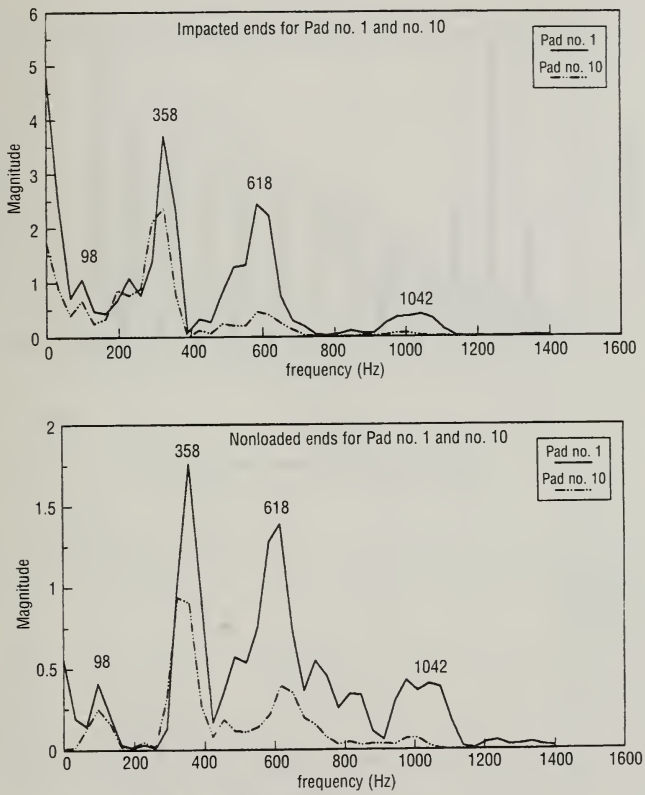


Figure 4. Frequency spectra of rail seat bending strain.

This result implies that a pad which can attenuate the tensile strain at the rail seat at one end does not necessarily attenuate the compressive strain at the other end. Although it is not a very common situation, it should be considered when a new pad is used for attenuating the impact load applied only at one end of tie, such as the impact due to rail abnormalities.

3. Impact load

The wheel-rail contact impact load may influence the working conditions of not only the ties and the track, but also the wheels and the train. In these tests, the impact load was measured with a load cell installed in the tup of the hammer. The average impact load attenuation for all pads is illustrated in Figure 9.

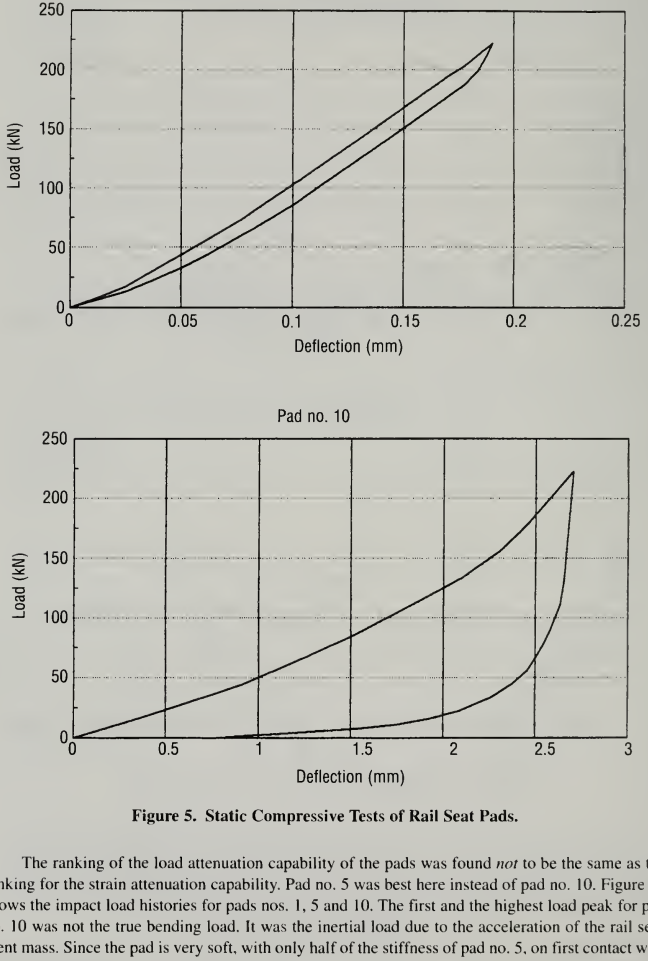


Figure 5. Static Compressive Tests of Rail Seat Pads.

The ranking of the load attenuation capability of the pads was found *not* to be the same as the ranking for the strain attenuation capability. Pad no. 5 was best here instead of pad no. 10. Figure 10 shows the impact load histories for pads nos. 1, 5 and 10. The first and the highest load peak for pad no. 10 was not the true bending load. It was the inertial load due to the acceleration of the rail segment mass. Since the pad is very soft, with only half of the stiffness of pad no. 5, on first contact with the striking tup the rail segment moved much quicker with pad no. 10 than with pad no. 5, leading to a much higher inertial load and a lower apparent impact load attenuation. However, comparing the true flexural loading of pad no. 10, which is the third peak of its curve, with the true flexural loading of pad no. 5, which is the second peak of its curve, pad no. 10 still behaved better with a lower flexural loading, corresponding to its better strain attenuation characteristics.

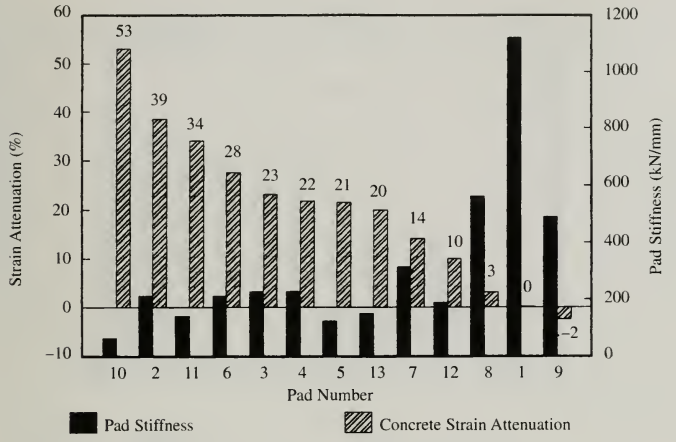


Figure 6. Pad performance under impact loading—Rail seat strain attenuation (Nonloaded End).

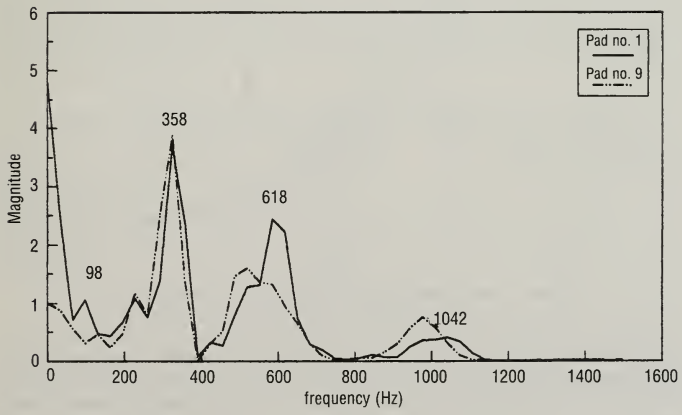


Figure 7. Frequency spectra of rail seat bending strain—Inpacted ends for Pads no. 1 and no. 9.

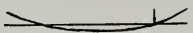
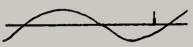
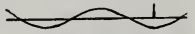
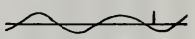
Mode	Mode Shapes	Measured Frequencies
1		98 Hz
2		358 Hz
3		618 Hz
4		1042 Hz

Figure 8. Mode Shapes and Frequencies of UP-2 Concrete Tie.

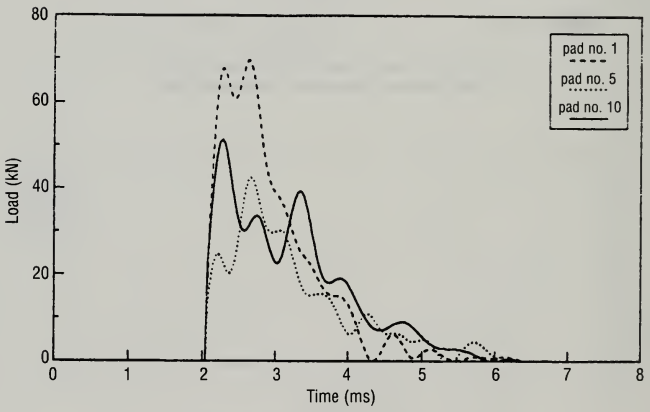


Figure 9. Impact load history for Pads no. 1, no. 5 and no. 10— Drop Height = 102 mm (4").

These results indicates that using different types of pads, the higher maximum wheel-rail contact load measured does not always lead to a higher tie response and vice versa. The inertial load may disturb the results. This should be given special attention when evaluating the properties of pads based only on the wheel-rail contact loads recorded.

4. Deterioration of static and dynamic pad characteristics

In this series of tests, two pairs of pads, with a new and a used pad for each pair, were checked. Pad no. 7 was the 6.5 mm EVA pad used for 185 MGT, and pad no. 9 was the 6.0 mm Lupolin flat EVA pad, used for 165 MGT; both of them having been in service for 3.5 years. Comparing the used

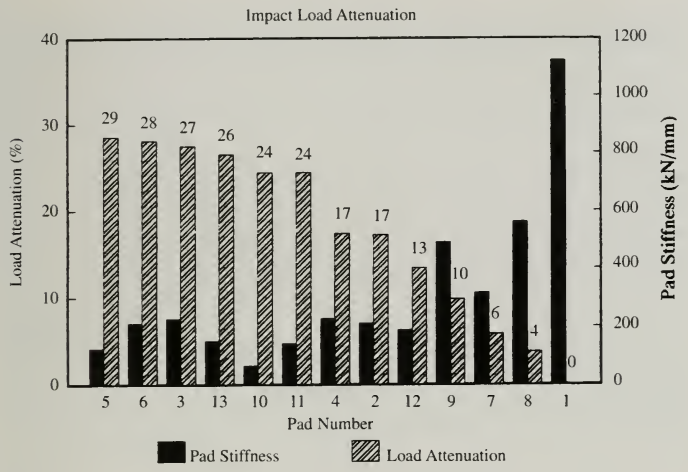


Figure 10. Pad performance under impact loading.

pad no. 9 with the new no. 8, it may be seen that the static stiffness, and the attenuation of rail seat strain at the impacted end and the nonloaded end are almost unchanged. But comparing the used pad no. 7 with the new pad no. 6, the changes are significant. Table 2 shows the changes of the properties for the two pairs of pads.

Table 2. Deterioration of used pad with respect to the same type of new pad

	Acme EVA pad	Lupolin flat EVA pad
Static Stiffness Increase	50%	0%
Impact Load Increase	30%	0%
Rail Seat Strain Response:		
Impacted end	10%	0%
Nonloaded end	19%	5%

Although the dynamic performance of a new Acme EVA pad no. 6, is much better than that of a new Lupolin flat EVA pad no. 8, after only 3.5 years in service, their performances had become very close to each other. An even worse influence of the Acme EVA pad than Lupolin EVA pad on ties might be predicted in subsequent years. In fact, a similar result was also obtained in the previous study[1], in which a moderately flexible type of pad behaved worse than the standard rigid EVA pad after being exposed to only 15 MGT tonnage in service.

The quick deterioration of pad no. 7 was due, at least partly, to the larger hysteresis loop in its static load- deflection curve showed in Figure 11. The pad may absorb more impact energy to protect the tie, but sacrificed itself in a reduced life. The correlation of the hysteresis loop in static-deflection curves of pads with the deterioration of pads under impact seems to merit further study.

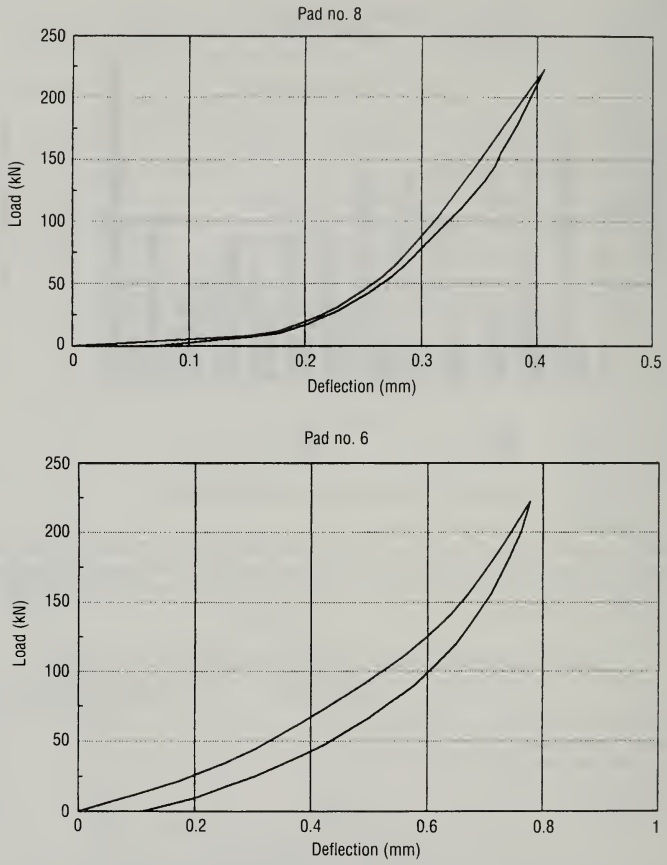


Figure 11. Static Compressive Tests of Rail Seat Pads.

Conclusions

A drop weight impact machine was built and fully instrumented. Using this machine, the impact pulse applied on a railroad track with concrete ties can be simulated. Thirteen types of pads installed on the tie using the Pandrol fastening system were tested and ranked. The attenuation effects of pads on impact load, tensile strain at impacted end and compressive strain at the nonloaded end were evaluated. A frequency domain analysis was carried out. Some of the results are:

1. Soft pads may act as a low-pass filter to significantly reduce energy in the second and the higher vibrating modes, leading to lower amplitudes but longer durations of the concrete strain vibration.
2. Pads that can attenuate the tensile strain at the impacted end do not necessarily attenuate the compressive strain at the other end. The frequency spectrum analysis may help to judge the properties of pads.
3. Using different types of pads under the same circumstances, a lower measured wheel-rail contact impact load does not always produce a lower rail seat strain response, and vice versa. The differing amounts of inertial loading may disturb the analysis.
4. In comparison with stiff pads, soft pads may structurally deteriorate more quickly, leading to an even worse influence on the ties after a period of time in service. The hysteresis loop in the static load-deflection curve of the pads might be used to predict the rate of deterioration. This topic merits further study.

Acknowledgments

The writers wish to acknowledge the support from ITISA Mexico who supplied the UP-2 concrete ties, and D. C. Wilcox of Pandrol Incorporated, New Jersey, who provided the components of the Pandrol fastening system. Thanks to J. F. Scott and J. A. Hadden for their personal help. Thanks are also due to Mr. M. Nazar and Mr. J. Wong for their preparation and maintenance of the impact machine and instrumentation.

References

1. Dean, F. E., Harrison, H. D., Prause, R. H. and Tuten, J.M., "Investigation of the effects of Tie Stiffness on the Impact Loading of Concrete Ties in the Northeast Corridor," Battelle Columbus Laboratories, *Report No. DOT/FRA/ORD-83/05*, April 1983.
2. Purbrick, M. C., "Track for the next decade—the mixed traffic railway," Track Technology, Thomas Telford Ltd, London 1985, pp. 44.
3. American Railway Engineering Association, *Manual for Railway Engineering*, 1993, Chapter 10.
4. Grassie, S. L., "Resilient railpads: their dynamic behaviour in the laboratory and on track," Proceedings Institution Mechanical Engineers, Vol. 203, Part F: *Journal of Rail and Rapid Transit*, 1989, pp. 25–32.
5. Igwemezie, J. O., and Mirza, M. S., "Impact load distribution in concrete bridge ties," *ASCE Journal of the Structural Engineering*, Vol. 115, No. 3, March 1989, pp. 526–542.
6. Venuti, W. J., "Report on Compressive Stiffness of Rail Seat Pads on Southern Pacific Concrete Ties," for Southern Pacific Transportation Co., San Francisco, California, April 1992.
7. Venuti, W. J., "Rail Displacements and Rotations," for Philip J. McQueen Corp., San Rafael, California, January, 1992.
8. Scott, J. F., "Wheel/Rail Impact Loading on Concrete Tie Track, Work Train Test," Canadian National Railways, *Report No. 191*, January, 1986.
9. Banthia, N., Mindess, S., and Bentur, A., "Impact Testing of Concrete Using a Drop-Weight Impact Machine." *Experimental Mechanics*, 1989, V. 29, n.1, pp. 63.
10. Ahlbeck, R. D., Tuten, J. M., Hadden J. A., and Harrison, H. D., "Development of Safety Criteria for Evaluating Concrete Tie Track in the Northeast Corridor," Battelle Columbus Laboratories, *Report No. DOT/FRA/ORD-86/08.1*, June 1986.

RESISTANCE OF CONCRETE RAILROAD TIES TO IMPACT LOADING

By: Nianzhi Wang*, Sidney Mindess* and William J. Venuti**

Introduciton

In June 1980, "hairline" bending cracks were noted under the rail seats of concrete ties in the Northeast Corridor track of Amtrak in the United States[1]. A visual inspection showed that as many as 50% of the total number of the ties in the sections checked were found to be cracked; some of the cracked ties had been in service for only a few months. The cracks were a cause of concern because the design service life for the ties was 50 years, and no cracks could be tolerated because of the harsh environment and possible fatigue of the prestressing steel at the crack gaps. Similar phenomena were later found elsewhere as well[2,3].

It became apparent that the primary cause of the cracking was due to impact loads arising from either "flats" on the rolling surface of the train wheels or to rail irregularities. Such loads could be up to 3 times higher than the normal quasi-static loads[1,3]. The measures usually taken in practice to combat this problem are: a) installing a rail seat pad with an increased resiliency (lower stiffness) between the rail and the tie rail seat to attenuate the impact force[1]; and b) detecting the flat wheels by means of a wheel impact load detector (installed on the rail) followed by changing or machining of the wheels[5]. These measures could lead to marked improvement. However, pads with a lower stiffness generally experience a decrease in fatigue life[6,7], leading to even more severe damage to the ties[1]. Also, changing wheel sets and sections of rail too frequently results in increased maintenance costs. An alternative approach is to evaluate the dynamic characteristics of concrete ties, with an eye toward improving their dynamic behavior such that impact forces are reduced.

The purpose of the study described here, which forms a part of a larger investigation of the behavior of prestressed concrete ties under both static and impact loading, was to examine the effects of the concrete compressive strength, steel fibre additions, changes in prestressing force, the presence of stirrups, and rail seat pad stiffness on the behavior of concrete ties under impact loading. This could, in turn, lead to improvements in the design of the concrete ties.

Experimental Procedures

Impact Machines

In order to simulate the response of concrete ties to impact loading, it was considered essential to replicate, as closely as possible, the impact pulse measured on a track in service, in terms of the duration of the pulse and the shape of the load vs. time curve. Two instrumented drop-weight impact machines, designed and built at the University of British Columbia, Canada, were used. The smaller machine, with a drop hammer mass of 60 kg, could produce an impact pulse with a 3–5 ms duration, which is considered to be the typical duration of the impact pulse due to wheel flats[1].This smaller machine is shown in Figure 1. The hammer weight, and the type and thickness of the rubber shim between the hammer and the striking tup are adjustable, and are thus capable of exciting any specified duration of pulse on the tie within certain limits. The drop height of the hammer influences the amplitude but not the duration of the pulse, as shown in Figure 2. The larger machine, with a drop hammer mass of 578 kg, could excite a 5–10 ms duration impact pulse on the ties[8,9], simulating the impact due to certain rail defects[4]. The load history for the impact testing on the tie, using the larger machine, is shown in Figure 3.

*Professor, Department of Civil Engineering, University of British Columbia, Vancouver, Canada
**Professor, Department of Civil Engineering, San Jose State University, San Jose, California

Figure 1. Instrumented drop-weight impact machine, 60 kg hammer.

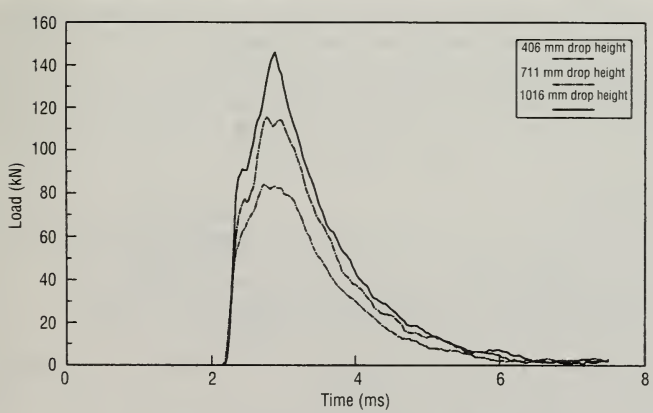


Figure 2. Impact Load Applied: Tie No. 1—Hammer Weight = 60 kg.

Specimens

Ties for this study were produced by ITISA of Mexico, based on the UP-2 post-tensioned tie which is produced for use in the United States. All dimensions were retained but the basic design was modified by changing the following parameters, either separately or in combination:

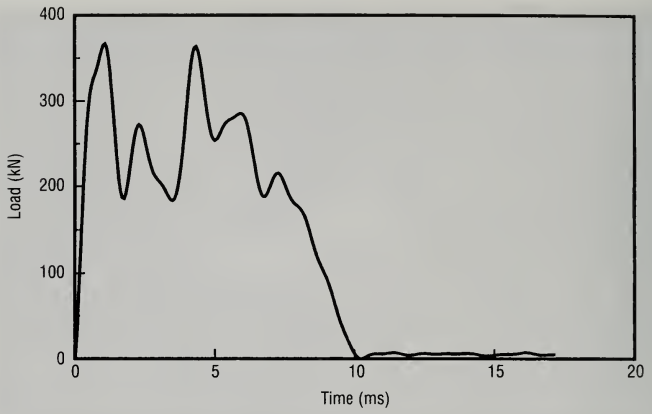


Figure 3. Impact Load History: Tie No. 1—Hammer Weight = 578 kg; Drop Height = 559 mm.

a) adding 1% by volume of 30 mm or 50 mm long steel fibres, of 0.5 mm diameter

b) adding seven stirrups of 5 mm diameter at 80 mm intervals under each of the two rail seats

c) reducing the concrete strength from 65 MPa to 40 MPa; and

d) reducing the prestressing force from 40,000 kg to 22,700 kg.

Details for each group of ties with the specimen numbers are given in Table 1.

Table 1: Concrete Tie Parameters

The Datum Tie

No. 1. Standard UP-2 Tie, Plain Concrete, $f'_{c\,=\,65\ MPa,\ 4-\phi 10.5\ mm\ prestressing\ bars\ and\ 40{,}000\ kg\ initial\ prestress\ force.}$ with an assumed prestress loss of 14%; the design bending resistance moment at the rail seat was 30.2 kN.m (267 inch-kips). They are referred to as "normal" ties in the discussion below.

Group 1: Additional steel fibres and stirrups, prestress unchanged at 40,000 kg.

No. 2. Plain Concrete + 0.5% 50 mm fibres, (40 kg/m^3 concrete).

No. 3. Plain Concrete + 1% 50 mm fibres, (79 kg/m^3 concrete).

No. 4. Plain Concrete + 1% 30 mm fibres, (79 kg/m^3 concrete)

No. 5. Plain Concrete + 7-ϕ5 mm stirrups for each rail seat at 80 mm intervals.

No. 6. Plain Concrete + stirrups + 1% 50 mm fibres.

Group 2: Changed prestressing bars and prestress level.

No. 7. Plain Concrete + ϕ9.5 mm prestressing bars and 22,700 kg prestress.

No. 8. Plain Concrete + ϕ9.5 mm bars and 22,700 kg prestress + 1% 30 mm fibres..

No. 9. Plain Concrete + ϕ9.5 mm bars and zero prestress.

No. 10. Plain Concrete + ϕ9.5 mm bars and zero prestress +1% 30 mm steel fibres.

Group 3: Changed concrete strength, prestress unchanged at 40,000 kg.

No. 11. Plain Concrete with lower strength, f'_c= 40 MPa.

No. 12. Plain Concrete with lower strength, f'_c= 40 MPa + 1% 30 mm fibres.

Test Methodology

The experimental setup for the impact test is shown in Figure 4. A standard 136 RE rail segment, 292 mm long, was fastened to the tie using a Pandrol fastening system with a 6.5 mm thick EVA rail pad between the rail and the tie. A detailed description of the fastening system used has been given elsewhere[9]. Two strain gauges*, 100 mm in length, were affixed to the tie below the two rail seats, 20 mm from the bottom of the tie, as shown in Figure 5. Crack detection gauges consisting of five 178 mm long silver lines painted longitudinally on the grounded concrete surface below the rail seat, 10, 35, 55, 75 and 95 mm from the bottom of the tie, were also used. They too are shown in Figure 5. These lines were connected to a resistance measuring circuit. As a crack grew, these lines were severed by the crack from the bottom line to the top line sequentially. Each cut caused the output of the circuit to jump to a new level, so that it was easy to determine which silver lines were cut. A small preload was used on the tie to prevent it from rebounding during loading. The details of the load cell in the drop hammer have been given in Reference 8.

For the first series of tests, carried out on the smaller impact machine, each tie was subjected to an accumulation of impact loads: one blow each for the drop heights of 102, 203 and 305 mm, and ten blows each for the drop heights of 406, 508, 610, 711, 813, 914 and 1016 mm. One tie for each tie number was tested except tie No. 12, for which the second tie was also tested to confirm its very good dynamic characteristics. For the second series of tests, using the larger impact machine, each tie was struck only ten times from a drop height of 559 mm. One tie for each tie number was tested. The total accumulated potential energies released for each tie in these two series are the same, for purposes of comparison. All of the data were collected at 0.03 ms intervals with a PC based acquisition system.

Results

For the first series of tests, the smaller machine with the 60 kg hammer weight was used. Figures 6, 7 and 8 show the maximum crack length and the residual crack length, for the different groups of ties. The vertical axis in these figures shows the vertical crack length. The horizontal axis shows the drop height, with the scale markings indicating the number of blows at a particular drop height. While the impact load was being applied, any crack that did develop opened to its maximum extent (referred to as maximum crack length), cutting one or more of the silver lines. After the impact event, because the crack was very fine, usually less than 0.05 mm in width, part of the crack near its

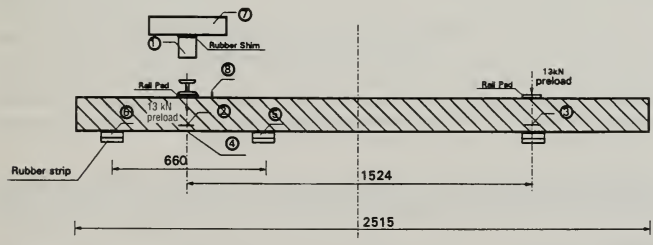


Figure 4. Layout of Impact Test for Concrete Railroad Ties. Ch. 1: load cell on tup; Ch. 2: strain gauge; Ch. 3: strain gauge; Ch. 4: crack gauge; Ch. 5: load cell on support; Ch. 6: load cell on support; Ch. 7: accelerometer on hammer; Ch. 8: accelerometer on tie. Stress coat applied on the opposite side of the tie at rail seats.

*Model EA-66-40CYB-120, manufactured by Measurements Group, Inc. USA.

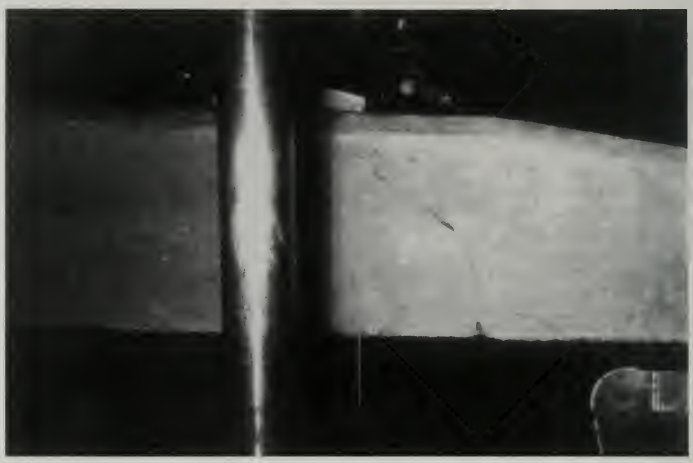


Figure 5. Typical crack pattern developed in a concrete tie under impact loading.

tip might be completely closed again due to the prestressing force, reconnecting some of the newly-cut silver lines and partially reestablishing the original output of the circuit. However, a portion of the crack would not close in the unloaded condition, and is referred to as the "residual" crack. The length of the residual crack is one of the parameters often used to evaluate the durability of concrete members in service. Both crack lengths, detected using the crack detection gauges, were measured from the bottom surface of the tie until the length reached 95 mm, which is the limit of measurement of the crack detection gauge, though cracks on some of the ties developed far beyond this limit such as tie No. 1 in Figure 6. The final crack width for each tie was measured using a hand-held graduated microscope with a 0.02 mm accuracy. The cracks were primarily "hair line" flexural cracks at the loaded end under the rail seat as shown in Figure 6, accompanied by some long horizontal cracks primarily located at the nonloaded end of the tie. For some ties, vertical cracks also occurred at the nonloaded end of the tie but were relatively shorter and finer. These cracks are not shown in the figures. A liquid coat of Stresscoat was also applied to the specially prepared concrete surface to locate fine cracks, and especially for measuring the length of the horizontal cracks.

Figures 9–11 are for the second series of tests, using the heavier drop hammer. Since the vertical cracks at the loaded end were very large, they could be measured easily with a scale. No horizontal or vertical cracks at the nonloaded end were found.

Discussion

1. Effect of steel fibres

The addition of steel fibres to the concrete mix greatly improved the tie behavior, resulting in shorter and finer cracks. Comparing ties No. 3 and No. 4 with tie No. 1 in Figure 6 and in Figure 9, we find that the tie with 30 mm fibres performed somewhat better in the first series of tests while the tie with 50 mm fibres performed better in the second series, although the differences were small. This is because the two machines induced different types of cracks on the ties. The shorter fibres have a

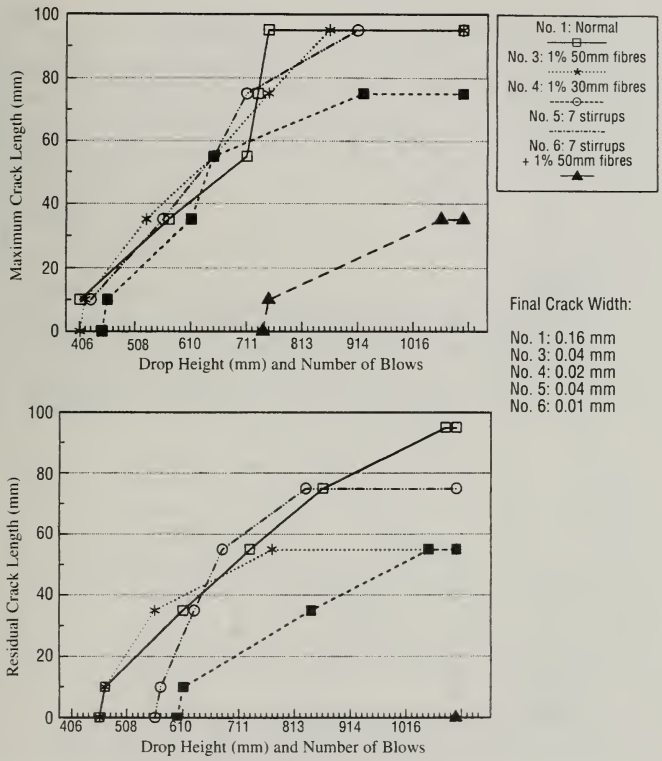


Figure 6. Fibre and Stirrup Effect on Crack Propagations—Crack Gauge Output; Hammer Weight = 60 kg.

smaller effective fibre spacing in the tie[10] and are therefore more effective in resisting microcracks and fine cracks; the longer fibres conceivably bridge across wider cracks, providing more effective resistance to the growth of wider cracks.

2. Effect of stirrups

Stirrups proved to be effective, particularly when used in conjunction with fibres in the first series of tests, as in tie No. 6, shown in Figure 6. However, they were less effective in the second series of tests when used in conjunction with fibres, as shown in Figure 9. One possible explanation is that stirrups resisted the growth of the horizontal cracks which occurred in the first series of tests, effectively retarding the deterioration of the concrete ties; they were not effective in resisting vertical cracking.

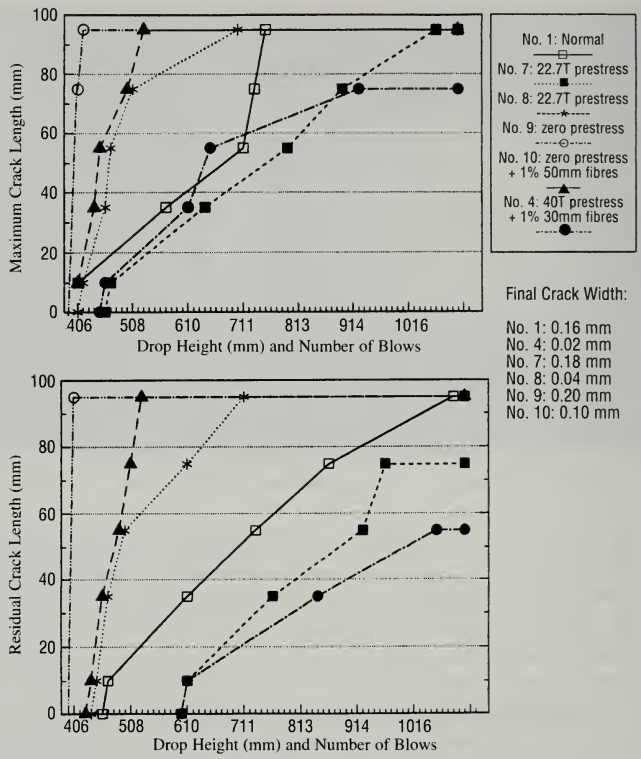

Maximum Crack Length (mm)
Drop Height (mm) and Number of Blows
Residual Crack Length (mm)
Drop Height (mm) and Number of Blows
No. 1: Normal
No. 7: 22.7T prestress
No. 8: 22.7T prestress
No. 9: zero prestress
No. 10: zero prestress + 1% 50mm fibres
No. 4: 40T prestress + 1% 30mm fibres
Final Crack Width:
No. 1: 0.16 mm
No. 4: 0.02 mm
No. 7: 0.18 mm
No. 8: 0.04 mm
No. 9: 0.20 mm
No. 10: 0.10 mm

Figure 7. Prestress Effect on Crack Propagations—Crack Gauge Output; Hammer Weight = 60 kg.

3. Effect of concrete strength

Tie No. 12 with a 40 MPa compressive strength and short fibres behaved excellently in both series of tests, as shown in Figures 8 and 11. Tie No. 12 was markedly better than tie No. 4, which had the same fibre content but 65 MPa concrete compressive strength. The reason for this may be seen in Figure 12. The lower stiffness of tie No. 12 caused the impact loads, the support loads and the concrete strains to be lower than those for tie No. 4. That is, the impact load applied to the tie depends not only on the drop weight and height of the hammer, but also on the properties of the tie itself.

Since the impact load on ties could be successfully simulated by a drop weight hammer striking a simply supported tie in the laboratory[15], a drop weight, *W*, and a simply supported beam were used as a simplified mechanical model to qualitatively analyze the relationship between the impact load and the mechanical properties of the beam.

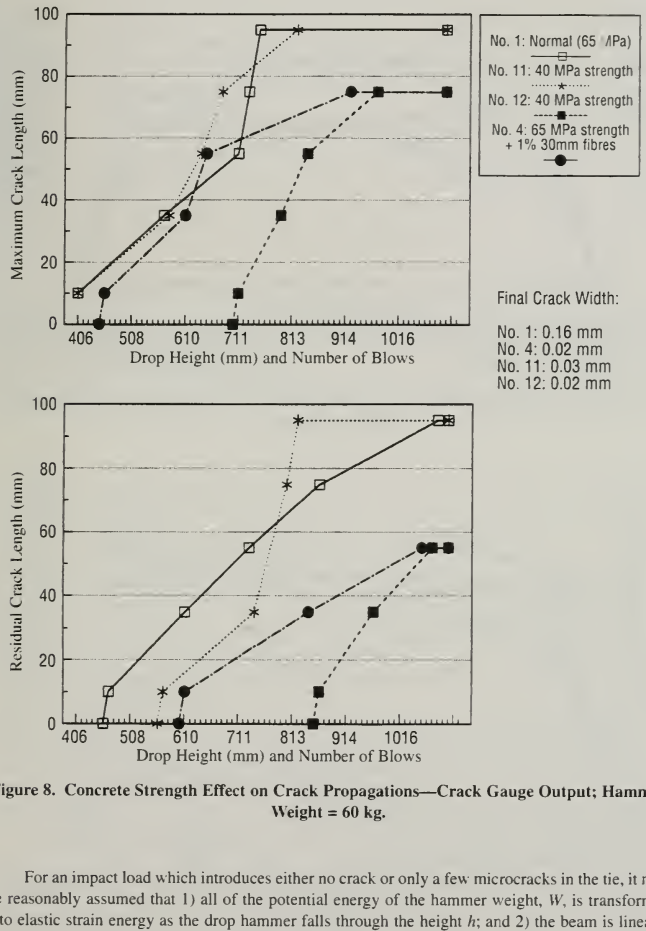

Maximum Crack Length (mm)
Drop Height (mm) and Number of Blows
No. 1: Normal (65 MPa)
No. 11: 40 MPa strength
No. 12: 40 MPa strength
No. 4: 65 MPa strength + 1% 30mm fibres
Final Crack Width:
No. 1: 0.16 mm
No. 4: 0.02 mm
No. 11: 0.03 mm
No. 12: 0.02 mm
Residual Crack Length (mm)
Drop Height (mm) and Number of Blows

Figure 8. Concrete Strength Effect on Crack Propagations—Crack Gauge Output; Hammer Weight = 60 kg.

For an impact load which introduces either no crack or only a few microcracks in the tie, it may be reasonably assumed that 1) all of the potential energy of the hammer weight, W, is transformed into elastic strain energy as the drop hammer falls through the height h; and 2) the beam is linearly elastic. From the law of conservation of energy, the equation

$$W(h+\Delta)-\frac{1}{2}F\Delta \qquad (1)$$

can easily be obtained[11], where Δ is the maximum deformation of the beam and F is the maximum force when W has reached its lowest point. The impact factor, IF, can be expressed as:

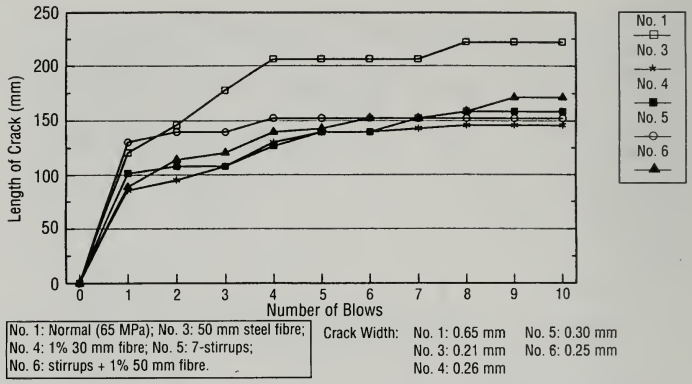

Length of Crack (mm)
Number of Blows
No. 1
No. 3
No. 4
No. 5
No. 6
No. 1: Normal (65 MPa); No. 3: 50 mm steel fibre;
No. 4: 1% 30 mm fibre; No. 5: 7-stirrups;
No. 6: stirrups + 1% 50 mm fibre.
Crack Width: No. 1: 0.65 mm No. 5: 0.30 mm
No. 3: 0.21 mm No. 6: 0.25 mm
No. 4: 0.26 mm

Figure 9. Fibre and Stirrup Effect on Crack Propagation—Hammer Weight = 578 kg, Drop Height = 559 mm.

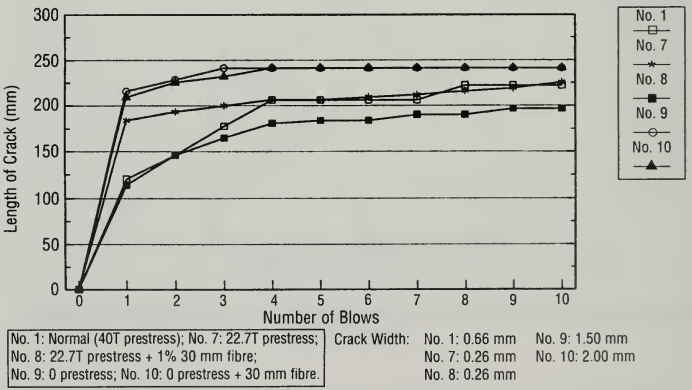

Length of Crack (mm)
Number of Blows
No. 1
No. 7
No. 8
No. 9
No. 10
No. 1: Normal (40T prestress); No. 7: 22.7T prestress;
No. 8: 22.7T prestress + 1% 30 mm fibre;
No. 9: 0 prestress; No. 10: 0 prestress + 30 mm fibre.
Crack Width: No. 1: 0.66 mm No. 9: 1.50 mm
No. 7: 0.26 mm No. 10: 2.00 mm
No. 8: 0.26 mm

Figure 10. Prestress Effect on Crack Propagation—Hammer Weight = 578 kg, Drop Height = 559 mm.

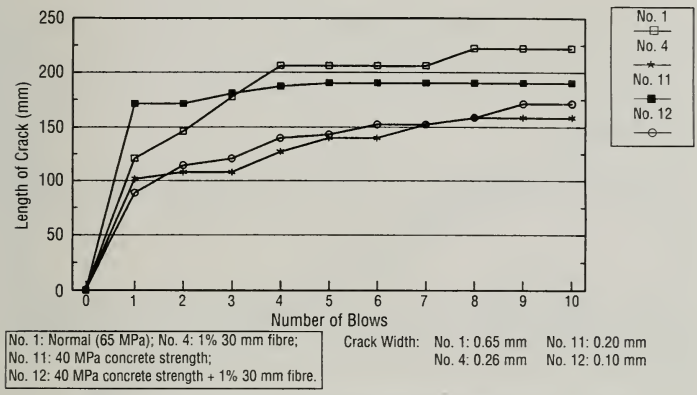

Length of Crack (mm)
Number of Blows
No. 1
No. 4
No. 11
No. 12
No. 1: Normal (65 MPa); No. 4: 1% 30 mm fibre;
No. 11: 40 MPa concrete strength;
No. 12: 40 MPa concrete strength + 1% 30 mm fibre.
Crack Width: No. 1: 0.65 mm No. 11: 0.20 mm
No. 4: 0.26 mm No. 12: 0.10 mm

Figure 11. Strength Effect on Crack Propagation—Hammer Weight = 578 kg, Drop Height = 559 mm.

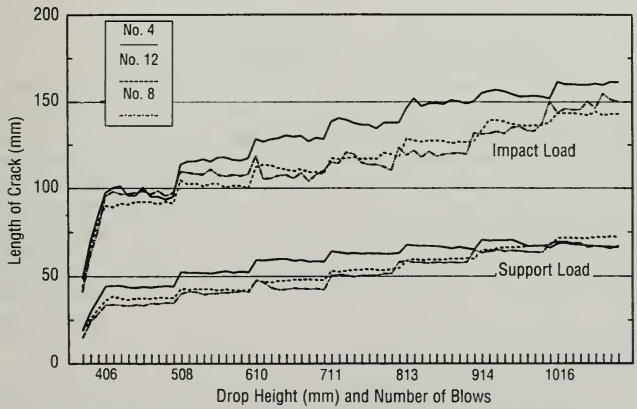


Figure 12. Maximum Load for Ties with 1% 30 mm Fibres—No. 4: 60 MPa; No. 12: 40 MPa; No. 8: low prestress.

$$IF = \frac{F}{W} = 2\left(1 + \frac{h}{\Delta}\right) \tag{2}$$

Using Δ = FL3/48EI for a simply supported beam, then IF in Equation 2 can be obtained as:

$$IF = 1 + \sqrt{\frac{96hEI}{WL^3}} \tag{3}$$

If the impact factor is very large, as it is in the present tests, for a specified testing system with constant drop weight W and support span L, Equation 3 may be simplified as

$$IF = \frac{F}{W} \propto \sqrt{hEI} \tag{4}$$

Moreover, since E is proportional to $\sqrt{f_c'}$, then IF is proportional to $f_c'^{0.25}$. Actually, in these tests, the maximum loads for different drop heights h, as shown in Figure 2, and the maximum loads for different concrete strengths of ties, as shown in Figure 12, are quite consistent with this formula. For example, we can calculate the ratios of the maximum loads at different drop heights:

$$\frac{F_{1016mm}}{F_{711mm}} = \frac{146kN}{117kN} = 1.25 \cong \frac{\sqrt{h = 1016}}{\sqrt{h = 711}} = 1.20 \tag{5}$$

and

$$\frac{F_{711mm}}{F_{406mm}} = \frac{117kN}{87kN} = 1.34 \cong \frac{\sqrt{h = 711mm}}{\sqrt{h = 406mm}} = 1.32 \tag{6}$$

Figure 13 shows that there are very good linear correlations between the maximum impact loads and the square roots of drop heights. The slopes of the regression lines for ties No. 1, 4, and 12 indicate the relative magnitude of the bending stiffness of each tie.

Equation 4 shows that a stronger but stiffer structure, with a higher EI, will incur a higher dynamic load, implying that the flexural strength is not the key factor in this type of dynamic design.

Evidence has shown[12,13] that the effective modulus for tracks made with wood ties may be as low as one sixth to one third of that for concrete tie tracks. That is why wood ties do not suffer from impact damage problems[12] although they do possess a lower flexural strength. Actually, the design impact factor in the design of the concrete ties has increased from 50%, which was recommended in the 1970's, to 200%, which is the current recommended value[13,14]. When higher impact loads were measured in track, the impact factor, and hence the concrete strength f_c' and stiffness EI were increased continuously to higher levels, leading to even higher impact loads on the "improved," stiffer ties. This was repeatedly followed by an increased impact factor.

Thus, the ductility, rather than the strength of the concrete, should be taken as the governing parameter, as is widely accepted in the design of earthquake resistant structures. The motivation for this series of tests, which included the consideration of the reduction of concrete strength and prestress force, while adding fibres and stirrups to increase the ductility of the ties, is based on this philosophy.

4. Effect of prestressing force

Three different prestress levels were used when making the concrete tie specimens. The normal UP-2 ties were post-tensioned using a 40,000 kg force. This prestressing force was reduced to 22,700 kg for ties No. 7 and 8; and there was zero prestress for ties No. 9 and 10, although the same size of

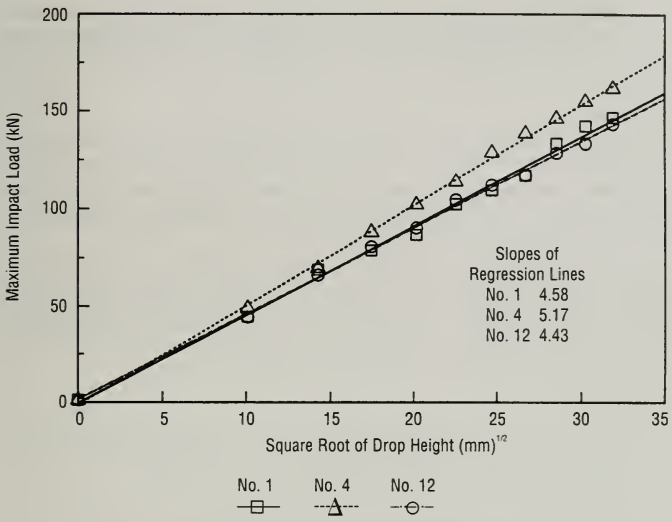


Figure 13. Relationship Between Impact Load and Drop Height.

steel bars was used for these ties. Figures 7 and 10 show that with zero prestress, ties No. 9 and 10 were damaged very quickly and severely, despite the addition of steel fibres in tie No. 10. However, tie No. 8, with approximately 50% less prestress, combined with short fibres, behaved very well, performing much better than tie No. 1 and similarly to Tie No. 4. The good quality of this tie is owing to, again, its responding with a relatively lower impact load, as shown in Figure 12, and its improved toughness. In another series of tests[16], it was also found that a lower prestress level in a beam may lead to a lower impact load applied by the drop hammer. The reason for that is not completely understood, but a previous study[17] has shown that the lower prestress level leads to a lower natural frequency and a lower dynamic flexural stiffness of concrete members.

Further Work

Keeping in mind that for prestressed concrete railroad ties, the current design impact factor is 200% and the design bending moment capacity is 3 times as high as the required quasi-static moment, reducing somewhat the moment of inertia of the tie, I, will not much influence the tie capacity for quasi-static loads, but may significantly reduce an applied impact load. For a beam with a rectangular cross section, substituting

$$I = \frac{bd^3}{12}$$

into Equation 4, we get the impact factor

$$IF \propto \sqrt{I} = \sqrt{bd^3/12} \propto b^{\frac{1}{2}} d^{\frac{3}{2}} \tag{7}$$

Reducing the depth d of the beam may reduce IF effectively. In addition, fibres and stirrups may be used in the tie to increase the ductility of the tie. Unfortunately, in this series of tests, we did not have the opportunity to modify the casting moulds in the plant so as to change the cross section of the ties.

Conclusions

1. Steel fibres greatly improved tie behavior, leading to shorter and finer cracks in concrete when subjected to impact loading.

2. Under repeated short duration pulses, the vertical cracks on the concrete ties may be accompanied by horizontal cracks. Stirrups can, particularly when used in conjunction with fibres, effectively retard the deterioration of the concrete tie.

3. Theoretical and experimental analyses have shown that the impact factor is proportional to the square root of both the drop height of the hammer and the flexural stiffness of the beam. Reducing the concrete compressive strength or prestressing level of the tie may result in a reduction of the dynamic flexural stiffness and hence the magnitude of impact loads. It is believed that if these measures were combined with the use of steel fibres in the concrete, a new type of concrete tie, with improved ductility and high resistance to impact load, could be realized.

References

1. Dean, F. E., Harrison, H.D., Prause, R.H. and Tuten, J.M.,"Investigation of the Effects of Tie Stiffness on the Impact Loading of Concrete Ties in the Northeast Corridor," Battelle's Columbus Laboratories, *Report No. DOT/FRA/ORD-83/05*, April 1983.

2. De Josez, B., "In-service testing of concrete bridge ties, Bridge 24.0 Winchester Subdivision," *CP Rail Report No. T-938-86*, 1987 Montreal, Quebec, Canada.

3. Purbrick, M.C., "Track for the next decade—the mixed traffic railway," *Track Technology*, 1985, pp. 44 Thomas Telford Ltd, London.

4. Scott, J. F., "Wheel/Rail Impact Loading on Concrete Tie Track, Work Train, Test Canadian National Railways," Transport Canada, *Report No. 191*, January 1986.

5. Ahlbeck, D. R., Tuten, J.M., Hadden, J. A., and Harrison, H.D., "Development of Safety Criteria for Evaluating Concrete Tie Track in the Northeast Corridor," Battelle's Columbus Laboratories, *Report No. DOT/FRA/ORD-86/08.*1 June 1986, Vol. 1, p. 34.

6. Watanabe, K., "Engineering of Rail Fastening," *Japanese Railway Engineering*, Vol. 19, No. 4, 1980.

7. FIP Commission on Prefabrication, "Concrete Railway Sleepers," *FIP State of Art Report*, Thomas Telford, London, 1987.

8. Banthia, N., Mindess, S., and Bentur, A., "Impact Testing of Concrete Using a Drop-Weight Impact Machine." *Experimental Mechanics*, 1989, V. 29, n.1, p. 63.

9. Ye, X., Wang, N., and Mindess, S., "Effect of Loading Rate and Support Conditions On the Mode of Failure of Prestressed Concrete Railroad Ties Subjected to Impact Loading," *Cement and Concrete Research*, Vol. 24, No. 7, p. 1286, 1994.

10. Bentur, A., and Mindess, S., *Fibre Reinforced Cementitious Composites*, Elsevier Applied Science, New York, 1990.

11. Muvdi, B. B., and McNabb, J.W., *Engineering Mechanics of Materials*, 3rd ed. Springer-Verlag, New York, Inc., 1991.

12. Driscoll, M. L., and Lamson, S. T., *Assessment of Steel Railway Ties*, Report No. 85-8, Canadian Institute of Guided Ground Transport, Queen's University, Kinston, Ontario, Oct., 1985.

13. Magee, G. M., "Prestressed Concrete Ties," *Railway Track and Structures*, Vol. 74, No. 8, August 1978, p. 18.

14. *Railway Track and Structure Encyclopedia*: Concrete tie, pp. S7-1, 1982

15. American Railway Engineering Association, *Manual for Railway Engineering*, 1993, Chapter 10.

16. Wang, N., and Mindess, S., "Impact Resistance of Post-tensioned Concrete," MRS Symposium, Boston, 1990, *Proceeding Volume 211, Fibre-Reinforced Cementitious Materials*, p. 149.

17. Grace, N. F., and Kennedy, J. B. "Dynamic Characteristics of Prestressed Waffle Slabs," *Journal of Structural Engineering*, Vol. 116, No. 6, June 1990, pp. 1547–1564.

CONRAIL FIELD TESTING OF THE ACOUSTIC EMISSION IN-PROCESS WELD MONITORING SYSTEM

By: C. P. Lonsdale* and Dr. M. N. Bassim**

Abstract

This study was conducted to determine if the in-process acoustic emission monitoring system could find thermite weld defects in a field testing situation on Conrail. Nine thermite welds were made out of track by Conrail welders at Alliance, Ohio and were evaluated by the system. The system, which has been used successfully with thermite welds on the Canadian National, collects, screens and analyzes post-weld acoustic emissions to determine if welds are sound.

Five Orgo-Thermit welds and four Boutet welds were produced. For each manufacturer one weld was produced to proper standards while the others were made to improper standards in order to induce deliberate defects. The welds were slow bend tested after production in order to facilitate fractographic examination and to obtain load and deflection data. The fractographic examination would determine if the welds were produced with significant defects present in their structures.

Significant defects were produced in four of the seven welds not made to proper standards. The acoustic emission system found three of these four welds. The system also correctly identified as sound the two welds made to proper standards containing no significant defects. Additionally, the system correctly identified two of the welds produced to improper standards as sound. Although it was expected that these two welds would be bad, they contained no significant defects. The acoustic emission system, therefore, exhibited an ability to differentiate between sound welds and welds containing significant defects.

Samples determined by fractography to be defective have lower average ultimate load and deflection values than samples determined to be sound. No such correlation can be made for the welds evaluated by acoustic emission means in this experiment.

Future work to explore and quantify the acoustic emission differences, if any, between welding products is planned. A baseline database of sound welds using 136 lb/yard rail with both Boutet and Orgo-Thermit welding material will be established. This will include a comparison and quantification of any differences between the new baseline and past system evaluations. Work to improve the portability, reliability, durability, ease of use and automation of the system will also continue.

Introduction

Consolidated Rail Corporation's interest in the application of an acoustic emission in-process thermite weld monitoring system began at the October 19, 1994, A.R.E.A. Committee 4 meeting. Dr. Nabil Bassim of Sysmon Corporation gave a brief presentation on the patented system developed for thermite railroad rail welding. The system is advertised to provide a pass/fail determination while the weld is cooling with no interpretive input from the operator[1]. Previous operational evaluations at CANAC International had proven successful and it was hoped that such a system could provide Conrail benefits as well[2,3].

In order to determine if the acoustic emission system for weld monitoring would perform accurately and efficiently in a "real railroad" environment, a test plan was developed. The plan, jointly devised by Mr. A. Pierson, Pittsburgh Division Welding Supervisor, and Mr. C. P. Lonsdale, Metallurgist at the Technical Services Laboratory, would test the system's ability to detect different

*Metallurgical Engineer, Conrail Technical Services Laboratory
**Sysmon Corp. President and CEO; Professor at University of Manitoba

types of weld defects. Additionally, the tests would not take place in a laboratory setting but would be conducted in a demanding, outdoor field environment. This would allow for a realistic test of the acoustic emission system under those conditions most often experienced by Conrail thermite welders.

If the in-process thermite weld monitoring system was deemed successful, it could be used to increase the safety and reliability of Conrail trackage. The acoustic emission system could be used by each thermite welding crew to determine the soundness of welds on a "go"/"no-go" basis. Alternatively, it could be used by Conrail welding supervision as a quality control and training tool.

Theory of Operation

Acoustic emission is a nondestructive testing technique which has been used, since the sixties, for assessment of extent of damage as well as for flaw location. It relies on the detection of stress waves occurring during deformation, cracking, or phase transformation. Acoustic emission differs from other techniques of nondestructive testing, because it is a passive technique which detects what occurs in the material and does not rely on sending an incident beam in the material.

Acoustic emission has been used to evaluate the quality of welds. In the welding and post-welding periods, changes, including solidification as well as phase transformations, cause significant acoustic emission to occur. One of the authors (M. N. Bassim) has studied the characteristics of such signals since 1978. He has found that a number of features in the signal, grouped by pattern recognition techniques, change markedly in the post-weld period for the sound (good) weld relative to the defective (bad) weld. Particularly, the amplitude distribution of the signals is reduced for the good weld and remains the same and even increases for the bad weld. This feature occurs after 15–23 minutes from welding for the thermite weld and is much shorter (1–3 minutes) for butt and submerged arc welds[4].

Thus, by following the changes in the signal features with time, it is possible to determine the quality of the weld. This approach was used successfully for over 50 thermite welds in a research program sponsored by Canadian National Railways at the University of Manitoba. In all cases, the technique lead to detection of the quality of welds as substantiated by ultrasonics and radiography tests as well as slow bend tests.

This feature was included in a patented automatic stand alone computer operated software controlled acoustic emission system, and has resulted in the production of the automated on-line weld monitor which is used in this investigation.

Scope of Evaluation

In order to determine how effectively the acoustic emission system performed in the field, the intent was to produce seven defective welds and two correct welds[5]. The first five welds were Orgo-Thermit products and the last four were from Boutet. Since Conrail uses both types of thermite welds, it was decided that products from both manufacturers should be used in the test procedures.

The test plan was designed to induce a specific type of weld defect into each "defective" weld. Defects in the base area were focused upon due to the difficulty in finding such weld defects with present means. The acoustic emission system would then have an opportunity to evaluate each weld as it cooled to ambient temperature. When the system completed its data acquisition and screening functions, the resultant determination of whether the weld was good or bad was compared against the intended nature of the weld. In order to confirm that a correct or defective weld had been produced independent of the monitoring system, the welds were slow bend tested. This destructive testing allowed examination of the fracture surfaces as well as gathering of load and displacement values.

The scope of our evaluation was limited to determining if the in-process weld monitoring system could find weld defects in a field test situation on Conrail. The slow bend testing was accomplished in order to facilitate a fractographic examination and to ascertain whether a "defective" weld had been made. The weld was determined to be defective if an induced defect was seen to be present

after fracture. Matching of welds that the acoustic emission system had labeled as "bad" or "good" could then be accomplished. It was also hoped that a correlation between load vs. displacement values and soundness of welds could be made.

Experimental: Weld Production

Five Orgo-Thermit SKV welds were produced out-of-track at the Conrail Alliance, Ohio Maintenance of Way Material Yard on November 29, 1994. Four Boutet welds were produced at the same location the following day. The weather was windy, sunny and about 52°F the first day. The second day's weather was cloudy and 42°F.

For the Orgo-Thermit welds, it was determined that weld number three would be made to proper specifications and the rest would be made deliberately defective. The intent with weld number one was to produce a weld with porosity in it due to excessive moisture from lack of crucible preheat. For weld number two, a narrow gap was used in order to create a weld with incomplete fusion at the base. Weld number four was exposed to a short preheat so that solidification contraction cracking might occur. Weld number five was made without a diverting plug so that slag would be entrapped in the weld matrix. Welds one, three and five were made using 140 lb/yard RE CC U.S. Steel Illinois 1968 rail. Welds two and four were made using 140 lb/yard RE CC U.S. Steel Illinois 1969 rail.

For the Boutet welds, all made with one use crucibles, weld number two was produced to proper specifications. The first weld was made with luting paste placed in the base area to cause an inclusion and/or an area of incomplete fusion. Weld number three was made with a narrow gap and short preheat in order to create a weld with incomplete fusion. The last Boutet weld was produced with the diverting plug removed in order to create a weld with entrapped slag. Welds one and three were produced with 132 lb/yard RE CC Bethlehem Steel 1971 rail. Welds two and four joined 132 lb/yard RE CC Inland Steel 1949 rail.

The Sysmon Corporation engineers set up their equipment for the acoustic emission monitoring system in the back of a van. Figure 1 is a diagram of the experimental setup. Figure 2 shows the equipment. Power for the computers, etc., was obtained by running an extension cord from a nearby work building. The van was backed up close to the welding site so that cables, running from the rail mounted transducers to the monitoring system, would reach. It should be noted that the Sysmon personnel were not informed which welds would be good or bad during the test. Also, every effort was made to make the test "blind" and not to give any indication as to which types of defects were being produced. This was done, along with the realistic pace and field environment, to fully evaluate the acoustic emission systems' capabilities.

The intent with the first Orgo-Thermit weld (O1) was to produce a weld with moisture induced porosity. It could then be determined if the acoustic emission system could find this type of defect. The molds were set up and left in place for forty-five minutes prior to the weld. No preheat of any kind was applied to the crucible or diverting plug. Prior to the beginning of the weld, two small spots were ground on each side of the rail web, centered between the head and base, approximately fifty cm from the weld center line. These locations are for the two acoustic emission transducers to be applied to the rail (with couplant) and attached with a "C" clamp. Figures 3, 4 and 5 show the location of the rail mounted transducers. During the test, the power in the work building was turned off by a departing, conscientious employee not aware of the welding test. As a result, the weld monitoring system was shut down and no information could be collected by the machine. Figure 6 shows a thermite weld reaction in progress.

The second Orgo-Thermit weld (O2) was produced with a 3/4 inch gap (normal=1 inch) in order to produce a weld with incomplete fusion at the base. All other welding parameters were maintained to specifications. This weld was produced quickly after the first weld as the welders were told to simulate actual "in track" conditions. As a result, the Sysmon engineers did not have enough time to attach the transducers to the rail prior to production of the weld. Since it is necessary to have the

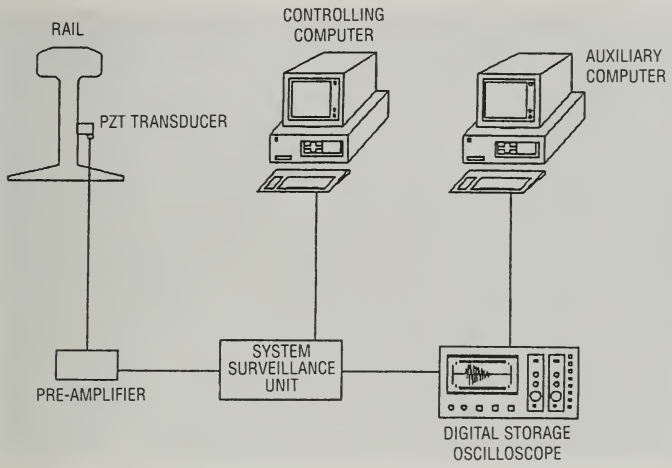


Figure 1. Experimental Setup.

Figure 2. Acoustic Emission System in Back of Vehicle at Test Site.

Figure 3. Sysmon Engineers Apply Transducers to the Rail with a C-Clamp.

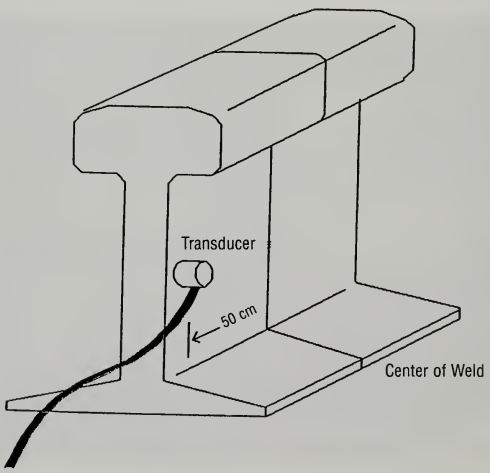


Figure 4. Transducer Position.

Figure 5. Single Transducer Attached to Rail with C-Clamp.

Figure 6. Orgo-Thermit Weld Reaction (to left of weld is spot ground for transducer).

transducers applied to the rail within about ten minutes after the weld in order to listen to the austenite to pearlite transformation, this was not deemed to be a problem. There was a slight delay while the van was backed up closer to the track since the transducer cable would not reach the far rail. Also, only one transducer was used for this test.

The third Orgo-Thermit weld (O3) was made properly with no defects deliberately introduced into the process. With weld O3 and all remaining welds, it was decided to put the transducers on the rail after the weld was poured and sheared. This was so that the clamp, transducers and cables would not interfere with or be damaged by the welders. Two transducers were used for this weld.

Orgo-Thermit weld number four (O4) was made with a very short rail preheat of 1.5 min. (normal=6 min.) with the hope of inducing solidification contraction cracking. During the pour of the weld metal from the crucible, it was noticed that a small leak had developed on the bottom of the rail base mold area. A small amount of liquid metal escaped the mold. One transducer was used to collect the acoustic emissions from this weld.

The diverting plug for weld number five (O5) was left out in order to cause a weld with entrapped slag. Some luting sand was also placed in the mold in the rail base area. All other welding parameters were performed to standard and one transducer was used for this weld.

The first Boutet weld (B1) on the second day of testing was produced with luting paste in the mold in the rail base area. As with all the Boutet welds, this one was produced with the "one shot" crucible. Also, due to better communication between Conrail and Sysmon personnel and application of "lessons learned" from the first day of testing, the operation proceeded in a much more organized, smoother fashion. Boutet weld number two (B2) was produced to proper standards with no induced defects.

The third Boutet weld (B3) was poured with a narrow gap of 3/4 inches and a short rail preheat of only three minutes. It was hoped that a weld with incomplete fusion and perhaps solidification cracking would be created. The acoustic emission system would then have an opportunity to detect this type of defect. The final Boutet weld (B4) was made without the diverting plug in place. The intended induced defect for this weld was slag entrapped in the weld matrix.

Acoustic Emission System Results

Table 1 contains the results of the acoustic emission monitoring system's evaluation, as well as the intended result of the welding process. The system's designation of a "bad" or "good" weld came after the acoustic signals were screened and processed by the computers and after output graphs were evaluated by Sysmon personnel. Two graphs are produced for each acoustic emission test by the system printer and these provide hard copy evidence regarding weld soundness.

Figure 7 is the first graph for sample O5, which was determined to be a bad weld. This curve shows a plot of amplitude of signals vs. time. A bad weld has a relatively horizontal band pattern through the center of the plot, as shown. Figure 8 is the second graph for sample O5. This curve shows a plot of percent peak signal distribution vs. time. A bad weld has the two plotted lines separated as shown.

Sample O4 is an example of a weld the acoustic emission system found to be good. Curves of amplitude of signals vs. time and percent peak signal distribution are shown as Figures 9 and 10 respectively. The horizontal band with the "bend" in the pattern is an indication of a sound weld, as shown in Figure 9. Figure 10 shows the two lines crossing each other. This also is an indication of a good weld.

As shown in Table 1, the acoustic emission system did not always agree with the intended result of the welding experiment. For example, welds O2, O4 and B4 were produced to improper standards so that a bad weld would be created. The monitoring system indicated that the welds were indeed sound. All other welds which were intended to be bad (O5, B1, B3) were deemed by the machine to

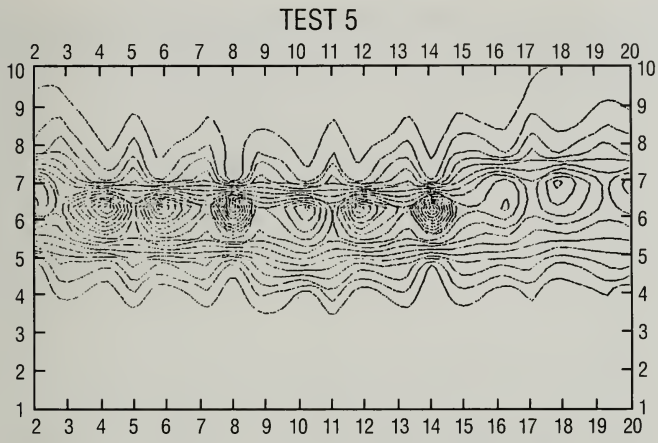


Figure 7. Acoustic Emission System Graph: Amplitude of Signals vs. Time for Weld O5.

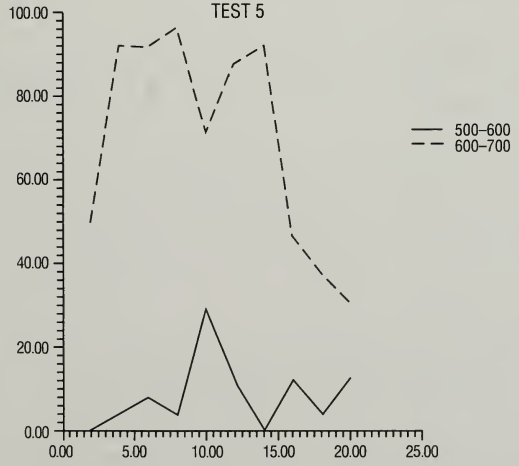


Figure 8. Acoustic Emission System Graph: Percent Peak Signal Distribution vs. Time for Weld O5.

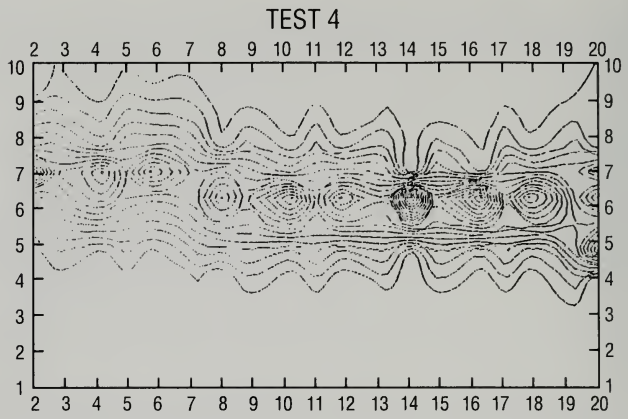


Figure 9. Acoustic Emission System Graph: Amplitude of Signals vs. Time for Weld O4.

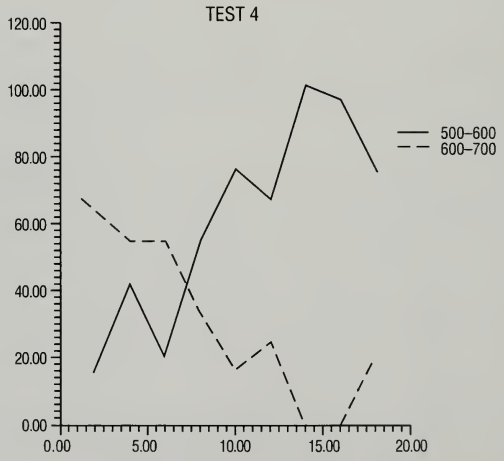


Figure 10. Acoustic Emission System Graph: Percent Peak Signal Distribution vs. Time for Weld O4.

Table 1. Acoustic Emission System Results

Weld Number	Intended Result	System Determination
O1	No Crucible Preheat—Bad	No Answer—Power Failure
O2	Narrow Gap 3/4 Inch—Bad	Good
O3	To Standards—Good	Good
O4	Short Preheat 1.5 min—Bad	Good
O5	No Diverting Plug & Sand in Base Area—Bad	Bad
B1	Luting Paste in Base—Bad	Bad
B2	To Standards—Good	Good
B3	Narrow Gap 3/4 Inch and 3 min. Preheat—Bad	Bad
B4	No Diverting Plug—Bad	Good

be bad (excluding O1 due to the power problem). The welds which were intended to be good (O3 and B2) were both found to be sound by the monitoring system. However, there still remained the issue of the three welds which had results which did not match. In order to resolve this discrepancy, the welds were to be slow bend tested. This would allow an examination of fracture surfaces to see if noticeable defects were present. It would also allow the collection of load and displacement data for each weld.

Slow Bend Testing

All nine welds were transported to Orgo-Thermit, Inc. at Lakehurst, NJ for slow bend testing. Each weld was centered in a five foot rail length. The slow bend test machine at Orgo-Thermit is routinely used for weld quality control procedures and normally operates in three point bending. The two points on the bottom are separated by a three foot distance and the top point is centered on the weld. This helps insure that the rail breaks in the weld area. All rails were loaded in a head up orientation. Although the primary goal of the slow bend test was to examine fracture surfaces of the weld to confirm or deny induced defects, load and deflection data was furnished by a machine plot. Figures 11 and 12 show the slow bend test machine.

Table 2 contains load and deflection data for the slow bend test. Figures 13 and 14 are graphs of the load and deflection data, respectively, for each weld sample. Figures 15 and 16 show the load and deflection data for each weld sample ranked in descending order. Values for the ultimate load and maximum deflection were obtained for a machine plot produced for each rail during the tests. Orgo-Thermit weld 2 (O2) fractured during the set up of the test when the press was brought down tight against the rail head. Therefore, zero load and deflection values resulted. All other rails were broken without adverse incident and values were obtained.

Fractographic Examination

Figures 17 through 38 are photographs of the weld sample fracture surfaces following slow bend testing. A fractographic examination of each surface was conducted in order to verify the soundness of each weld. Table 3 summarizes and compares the fractographic and acoustic emission system results.

Weld O1 exhibits a granular fracture surface consistent with brittle fracture. No plastic deformation is evident. The sample fractured at the edge of the weld in the rail/weld interface area. One small area of porosity is noted at the top of the rail base near the end. The round pore measures approximately two mm in diameter and was not the fracture origination point. Fracture proceeded from the base to the head as was expected. Considering the location and the small size of this pore, ultrasonic testing would not have found this weld unserviceable. This weld shows no fractographic evidence of being a bad weld despite the intent to produce one during the experiment.

Figure 11. 3 Point Slow Bend Test Machine.

Figure 12. Slow Bend Test Machine Close-Up Prior to Start of Testing.

Table 2. Slow Bend Test Data

Weld	Load (x1,000 lb)	Deflection (Inches)
O1	335.7	0.425
O2	0*	0*
O3	320	0.34
O4	347.1	0.42
O5	331.4	0.33
B1	182.9	0.17
B2	348.6	0.37
B3	368.6	0.42
B4	327.1	0.34

O = Orgo-Thermit
B = Boutet
*Broken during set-up of slow bend test, zero load and deflection.

Weld O2 shows that the failure originated in the middle of the web at one surface. It fractured at the edge of the weld in the rail/weld interface area. When this sample was slow bend tested, it failed during the machine set up with a very small load applied to it. The fracture surfaces indicated that only about 25% of the weld metal was sound and that the weld had cracked internally prior to bend testing. This weld is classified as a bad weld, as was the production intent.

Weld O3's fracture originated at the base of the rail/weld joint area. There is no evidence of porosity, voids or incomplete fusion. The fracture surface is granular, shows no plastic deformation and indicates that brittle rupture occurred. This weld shows no evidence of being a bad weld and the original intent was to produce a sound weld.

Two small pores and two small slag inclusions are evident on the fracture surfaces of weld O4. One pore is located at the bottom of the base directly under the web and is approximately two mm in diameter. The other pore is at the top of the base at the web/base fillet and is about one mm in diameter. The two inclusions are very small and there is one on each side of the web at the edge. This weld, which fractured at the base in the rail/weld interface area, shows a granular fracture surface with no plastic deformation. Fracture proceeded from the base in a brittle fashion. The defects in this weld are very small and this weld is therefore classified as a sound weld. It is doubtful that these defects would have been found by ultrasonic inspection. Our intent had been to produce a bad weld.

For weld O5, the entire head of the weld and about one third of the web is filled with slag on the fracture surfaces. The balance of the fracture surfaces are granular and consistent with brittle fracture. There are no defects apparent in the base. This weld is clearly a bad weld as was intended in the experiment.

Weld B1 fractured at the base in the weld, but near an edge of the weld. This weld's fracture surfaces show two areas of incomplete fusion along the bottom of the base. One defect is approximately eight mm wide and sixty-three mm long. The other defect is smaller; two mm wide and four mm long. Another small defect is seen on the side of the rail head measuring one mm wide and three mm long. Crack chevrons lead back towards the end of the largest defect indicating that fracture originated there. The fracture surface exhibits the features of a brittle fracture. This weld is clearly a bad weld as was the production intent. Due to the location of the defects, an ultrasonic inspection would not have found this weld to be bad.

The fracture for weld B2 originated at the base at the edge of the weld in the rail/weld interface area. At the far end of the base at the bottom a small void is evident. This vertically oriented void measures approximately one mm wide and three mm long. The weld fractured in a brittle manner

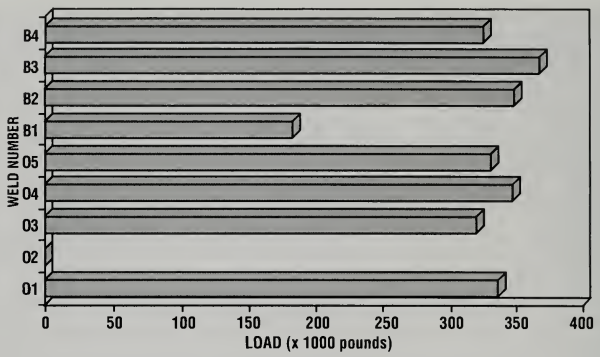


Figure 13. Slow Bend Test—Ultimate Load Data.

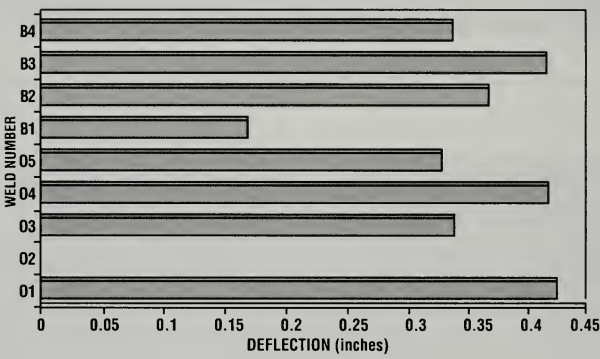


Figure 14. Slow Bend Test—Maximum Deflection Data.

with no plastic deformation evident. The defect in the weld is quite small and the weld is therefore classified as a sound product. Such a defect would not be found by ultrasonic testing. This result matches the intent of producing a good weld.

The third Boutet weld, B3, has fracture surfaces which exhibit the most chevrons and deformation. The weld fracture originated in the weld at the base in the rail/weld interface area. A large transverse section broke out of the weld during fracture. This piece is composed of the entire cross sectional area of the web and two thirds of the base. An area of slag is evident at one extreme end of the base in this section which separated. This area of slag is "L" shaped following the contour of the bottom of the base and the foot of the rail. The discontinuity extends twenty mm along the bottom of the base and twelve mm upwards along the foot of the rail. Fracture originated at this defect. This

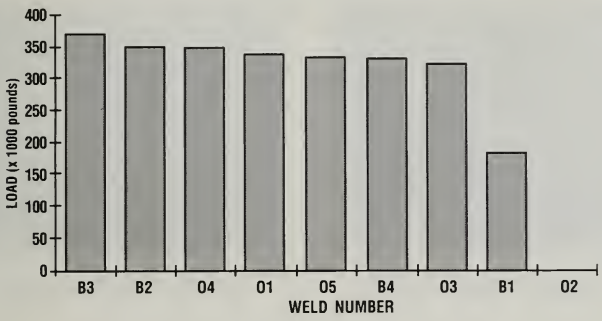


Figure 15. Ultimate Load—Descending Order.

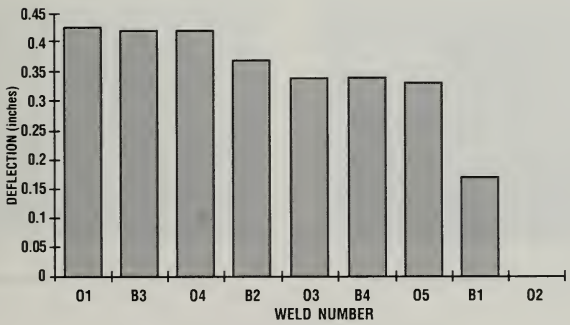


Figure 16. Deflection—Descending Order.

weld is classified as a bad weld as was the intended welding result. Ultrasonic flaw detection would not have found this defect due to its location.

The final sample, weld B4, fractured at the edge of the rail/weld interface with the origination at the base. The fracture surface is granular with no plastic deformation and, therefore, shows evidence of brittle rupture. A small void seen along the edge of the fracture surface in the collar area measuring four mm by twenty mm would have been ground away under normal welding practice. This weld is classified as a sound weld due to the lack of apparent defects, although the intent was to create a defective weld.

Discussion

As previously indicated, the acoustic emission system indicated that welds O5, B1 and B3 were bad welds and welds O2, O3, O4, B2 and B4 were sound (O1 not evaluated). Prior to the slow bend

Figure 17. O1 Fracture Surface.

Figure 18. O1 Fracture Surface.

Figure 19. O2 Fracture Surface.

Figure 20. O2 Fracture Surface.

Figure 21. O3 Fracture Surface.

Figure 22. O3 Fracture Surface.

Figure 23. O4 Fracture Surface.

Figure 24. O4 Fracture Surface.

Figure 25. O5 Fracture Surface.

Figure 26. O5 Fracture Surface.

Figure 27. B1 Fracture Surface.

Figure 28. B1 Fracture Surface.

Figure 29. B2 Fracture Surface.

Figure 30. B2 Fracture Surface.

Figure 31. B3 Fracture Surface.

Figure 32. B3 Fracture Surface.

Figure 33. B4 Fracture Surface.

Figure 34. B4 Fracture Surface.

Figure 35. Defective Weld O2.

Figure 36. Slag from Rail Head, Weld O5.

Figure 37. Defective Weld B1, Close Up of Base Defect.

Figure 38. Defective Weld B3, Portion Broken Out, Defect Lower Right Corner.

Table 3: Summary of Fractographic and Acoustic Emission System Results

Weld Number	Intended Result	System Determination	Actual Results (*)	System Match With Actual Result?
O1	Bad	Not Available	Good	Not Available
O2	Bad	Good	Bad	No
O3	Good	Good	Good	Yes
O4	Bad	Good	Good	Yes
O5	Bad	Bad	Bad	Yes
B1	Bad	Bad	Bad	Yes
B2	Good	Good	Good	Yes
B3	Bad	Bad	Bad	Yes
B4	Bad	Good	Good	Yes

*Determined by fractographic examination.

tests and fractographic examination, welds O2, O4 and B4 were samples where the intended production result did not agree with the acoustic emission system. The results of the fractographic examination indicated that welds O1, O3, O4, B2 and B4 were sound, whereas welds O2, O5, B1 and B3 contained significant defects. The acoustic emission system did not, therefore, find one of the defective welds, sample O2. In addition to finding three of the four bad welds, the system discovered that two welds we intended to make defective were good (O4 and B4).

A possible reason for the system not finding O2 to be defective is the loss of power during the evaluation of weld O1. It is possible that the system did not reset properly and, therefore, did not identify the problems with O2. It is interesting to note that welds O4 and B4 would have been considered "scrap" without the system's evaluation. To make a weld with an extremely short preheat (and leak) or without a diverting plug is not proper procedure and is grounds for weld removal. That the welds were found to be good points to both the somewhat forgiving nature of thermite welding and the ability of the acoustic emission system.

With respect to the load and deflection data, it is seen that O2 has the worst load and deflection values at zero. Weld B1 has the second lowest values for load and deflection. The other two welds noted as defective by the acoustic emission system and fractography, O5 and B3, have higher values. Weld O5 has the third lowest deflection and fifth lowest load despite having slag throughout the head. It is concluded that these O5 values would be much lower if the rails were tested head down. Finally, weld B3 has excellent load and deflection values. This weld has the best ultimate load and second best deflection. Though found defective by the system and post bend test fractography, this weld may have performed satisfactorily in service. It would be susceptible to fatigue crack initiation due to the stress concentration at the noted discontinuity, however.

After a categorization of welds as sound or defective by fractographic examination, average ultimate load and deflection values by category were compared. Table 4 contains this data. It can be seen that sound welds as a group have significantly greater load and deflection values than the defective welds. Due to the zero load and deflection values of sample O2, average values of the defective group were calculated both including and excluding O2. When O2 is excluded, the average values for defective welds are still significantly lower than the values for sound welds.

Table 4: Average Ultimate Load and Deflection Values by Category— as Determined by Fractography

Category—Determination Based On Fractographic Exam	LOAD (x 1,000 lb)	DEFLECTION (Inches)
Sound Welds	335.7	0.38
Defective Welds (Incl. O2)	222.5	0.23
Defective Welds (Excl. O2)	296.6	0.31

Table 5: Average Ultimate Load and Deflection Values by Catergory— as Determined by Acoustic Emission System

Category—Determination Based On Acoustic Emission System*	Load (x 1,000 lb)	Deflection (Inches)
Defective Welds	294.3	0.31
Other Welds (Incl. O2)	268.6	0.29
Other Welds (Excl. O2)	335.7	0.37

*O1 not included in this table.

Ultimate load and deflection values by category as determined by the acoustic emission system are contained in Table 5. Due to the fact that the acoustic emission system missed defective weld O2, the values for defective welds are higher than the values for other welds not identified as defective by the system. If weld O2 is not included in the averages, the welds found to be defective have lower load and deflection values. This latter comparison is not intended to minimize the inability of the acoustic emission system to find the obviously defective weld O2, however. For all data in Table 5, weld O1 was not included since the system did not have an opportunity to evaluate it.

At present, the acoustic emission system is not entirely portable and user friendly. The system is made up of a group of components which require electric power. Also, some interpretation of the results is indeed required by the operator. Future efforts by Sysmon Corporation will be directed towards making the weld monitoring system more portable and easy to use by welding crews in the field. Such efforts should be centered on durability, reliability and complete automation in a portable system.

During the course of testing Orgo-Thermit welds the first day, Sysmon engineers mentioned that the data output was different than they had seen on previous Canadian tests. All previous tests performed by Sysmon had been done on Boutet products. When testing of the Boutet welds began the second day, the Sysmon personnel indicated that the output more resembled the Canadian tests. It was postulated that there may be some calibration effect between the different weld types. Since Conrail uses both Orgo-Thermit and Boutet welds, any differences in output should be identified and quantified to insure inspection reliability. Also, in order to help increase testing efficiency and effectiveness, a series of tests using 136 lb/yard standard and fully heat treated rail with both Boutet and Orgo-Thermit welds needs to be completed. This will allow for the establishment of a Conrail standard "calibration baseline" for good welds. Sysmon Corporation has agreed that this has merit and would be beneficial to improve testing accuracy.

Conclusions

1. The experiment designed to test the acoustic emission weld monitoring system succeeded in creating significant defects in four of the seven welds not made to proper standards (welds O2, O5, B1 and B3).

2. The acoustic emission system indicated that two welds (O4 and B4) produced to improper standards in order to induce defects were sound. These welds were expected by Conrail to be bad, but contained no apparent significant defects after slow bend testing.

3. The system correctly identified both welds produced to proper standards (O3 and B2) as sound welds. These welds contained no apparent significant defects after slow bend testing.

4. The system found three of the four welds containing significant defects to be bad welds. Weld O2, which was clearly seen to be a bad weld after slow bend testing, was not identified as bad by the system.

5. For the welds examined in this test, samples determined to be defective by fractographic examination have lower average ultimate load and deflection values than samples determined to be sound. No such correlation can be made for welds evaluated by acoustic emission means in this experiment.

6. The acoustic emission weld monitoring system has exhibited an ability to differentiate between sound welds and welds containing significant defects. The system did so in a field testing environment.

Recommendations

In order to provide for maximum thermite weld evaluation efficiency and effectiveness using the acoustic emission system:

1. The acoustic emission differences, if any, between different welding products should be explored and quantified.

2. A baseline database of sound welds using 136 lb/yard standard and fully heat treated rail and both Boutet and Orgo-Thermit welding material should be established. This would include a comparison and quantification of any differences between the new baseline and past system evaluations.

3. Planned system improvements centering on portability, reliability, durability, ease of use and automation should be continued. The railroad field employee who will use the system should be considered when making all design improvements.

References

1. Sysmon Corporation, "Sysmon Weld Test—An Acoustic Emission Expert System For In-Process Weld Monitoring." Sysmon Corporation, Winnipeg, Manitoba, Canada.

2. Personal communication with Dr. John Strasser, CANAC International, Inc., December 29, 1994.

3. CANAC International, Executive Summary of Internal Report on Acoustic Emission Tests, personal communication with Dr. John Strasser, CANAC International, Inc., December 29, 1994.

4. Bassim, M. N., Hay, D. R. and Talerman, M., "Postweld Evaluation By Acoustic Emission," Materials Evaluation, May, 1979.

5. Conrail Corporation, *MW-125 Manual For Welding and Grinding-Maintenance of Way*, Conrail, Philadelphia, PA.

MEMOIR
Earl T. Franzen
1915–1995

After a lengthy illness, Earl T. Franzen died on May 16, 1995 in Kirkwood, Missouri at the age of 80. He resided in Webster Groves, MO. Earl was born in Palisade, MN and attended the University of Minnesota, where he earned a degree in Civil Engineering.

Earl joined the Great Northern Railway in 1937 upon his graduation, where he worked in the Bridge Department. He subsequently was employed as a bridge engineer by the Wisconsin Highway Department and the Rock Island Railroad, prior to accepting a position with the Missouri Pacific Railroad in 1967. At the time of his retirement from the MoPac in 1982, Earl was Chief Engineer—Design and Construction.

Earl joined the AREA in 1942 and was designated a Life Member in 1978. He will be missed by all of his associates and friends, who knew and worked with him in the railroad industry. Earl is survived by his wife, Irene, three sons and four grandchildren.

MEMOIR
Robert E. Kleist
1923–1994

Robert E. Kleist died October 14, 1994 in Philadelphia, Pennsylvania after a long illness. He was 71. Bob retired from the Association of American Railroads in January 1990, after serving the railroad industry for 44 years. He is survived by his wife, Arlene, two daughters, Donna Lyons, and Diane Livingston and four grandchildren.

Born in Milwaukee, Wisconsin, Bob graduated from Boys Technical High School in 1942. He was drafted into the U.S. Navy's V-12 Program in 1943, and honorably discharged in 1946. While in the Navy, he attended the University of Washington and graduated as a Civil Engineer from the Case School of Applied Science, Cleveland, Ohio in 1946. In 1958 he received his Professional Engineer's license in New York.

Bob started his railroad career as a Junior Engineer on the Pennsylvania Railroad in Mansfield, Ohio. He later held positions as Assistant Track Supervisor at Columbus, Ohio and Wilmington, Delaware; Track Supervisor at Orrville and Alliance, Ohio, Terre Haute, Indiana and Wilmington, Delaware. In 1956 he became Assistant District Engineer in Buffalo, New York. In 1958, Bob was appointed Assistant Engineer of Tests, Altoona, Pennsylvania. In 1962, Bob became Assistant District Engineer in Pittsburgh, PA.

Bob joined the Railway Maintenance Corporation as General Sales Manager in 1962.

He was employed by the Federal Railroad Administration's Safety Department in Philadelphia in 1973. In 1975, Bob moved to FRA's Washington, D.C. office where he was involved with the redevelopment of the Northeast Corridor for high speed rail service. He then served in the Office of National Freight Assistance Programs.

From 1982 until his retirement in 1990 he was Executive Director of the Systems Studies Division of the Association of American Railroads in Washington, D.C. After retiring, Bob was a consultant for both the AAR and the Transportation Research Board.

Bob was active in the Transportation Research Board. He was a member of the Group 2 Council for five years, chairman of the Railway Systems Section for six years, and chairman of the committee on Track Structure System Design for six years.

Bob joined AREA as a junior member in 1948, became a member in 1954 and was awarded life membership in 1989. He served on Committee 22—Economics of Railway Construction & Maintenance, Committee 27—Maintenance of Way Work Equipment and Committee 17—High Speed Rail since its inception in 1988.

He was a member of the Lutheran Church where he held various offices, was a member of the choir, taught Sunday school and counseled youth groups.

Bob's many interests and activities included playing the cello, furniture refinishing, gardening, and the study of birds.

Bob was held in high esteem by his many co-workers and associates. His professionalism, integrity, keen sense of humor, generosity, his joy of life, courage in the face of illness, his diplomacy, kindness, reassuring calm in time of stress and his deep faith in God will all be remembered and sorely missed by all of us who knew him.

MEMOIR
Hugh Bradley Lewis
1922–1995

Hugh B. Lewis, age 73, retired Associate and Structures Manager, Riley, Park, Hayden & Associates, Inc., died August 13, 1995 at his home in Leesburg, Florida. He is survived by his wife, Barbara, two children, Mark B. Lewis and Kathleen Sokolitsky, and five grandchildren.

Mr. Lewis was born January 12, 1922 in Shreveport, Louisiana. He began his railroad career with the Louisville & Nashville Railroad Co. as an instrumentman and draftsman after graduating from the Georgia Institute of Technology, where he earned a Bachelor of Science degree in Civil Engineering in 1943. During his 30 years with the L&N, he advanced through a number of assignments until he became an Assistant Bridge Engineer with the responsibility for the maintenance, design, construction and improvements to all drawbridges on the system. Upon leaving the L&N in 1973, he joined the staff of Riley, Park, Hayden & Associates, Inc. as Bridge Engineer and continued his railroad career with the design of railroad relocation projects, intermodal facilities and classification yards in addition to numerous bridges. He advanced quickly and was named an Associate in the firm and promoted to Manager of the Structural Department, a position he held when he retired in 1987.

He was an active member of the American Railway Engineering Association, serving on Committee 8—Concrete Structures and Foundations and Committee 15—Steel Structures. He was also a member of the National Society of Professional Engineers, the American Society of Civil Engineers, the American Welding Society, the American Concrete Institute and Chi Epsilon. He was a Registered Professional Engineer in the states of Georgia, Indiana, South Carolina and Tennessee.

Mr. Lewis' long and distinguished service to the railroad industry, the AREA and former employers will be sadly missed and well remembered by those of us who worked with him.

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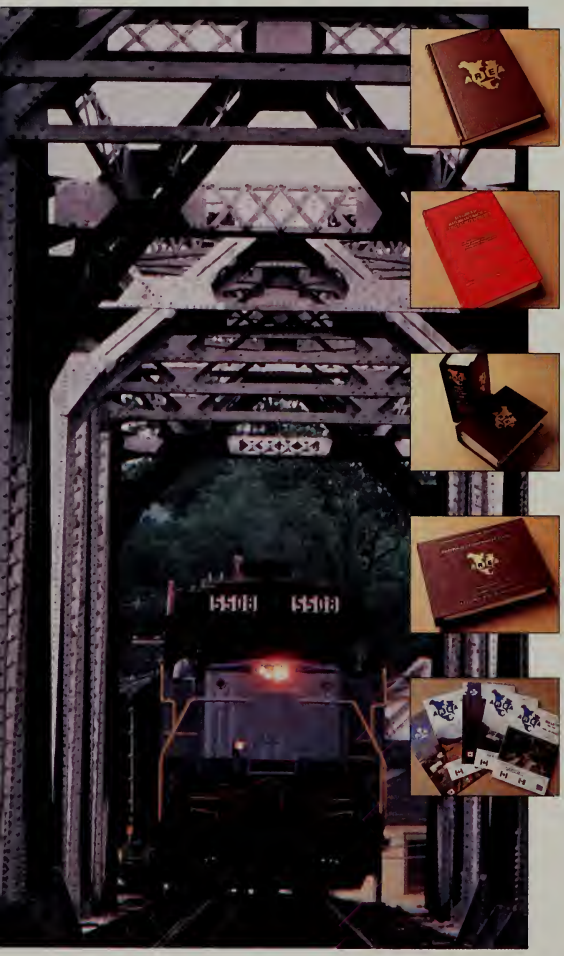

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American Railway Engineering Association
50 F Street, N.W.
Washington, D.C. 20001
(202) 639-2190

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